HIGHWAY RESEARCH RECORD

Number 378 | Nondestructive Testing of Concrete

7 reports

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Subject Areas

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FOREWORD

Advancements in concrete technology and in testing and quality control of concrete have moved to the forefront the problem of determining the quality of concrete in structures, the so-called in situ quality. Although several destructive methods, such as breaking separately molded concrete specimens or taking out specimens from finished structures, have been used for this purpose, it has become obvious that only nondestructive methods provide sensible means for testing concrete in structures. Unfortunately, the currently used nondestructive methods, such as pulse velocity and Schmidt test hammer, do not provide results that are accurate enough. It seems that a considerable amount of future research is needed to adequately improve the accuracy of nondestructive testing for concrete.

The 7 studies reported in this RECORD represent advancements in nondestructive testing. Thus, they contribute directly or indirectly to solutions of the problem of in situ testing of concrete.

The paper by Li, Ramakrishnan, and Russell is a state-of-the-art report concerning both field and laboratory methods for nondestructive testing of concrete. The various uses of ultrasonic techniques for concrete are discussed in the papers by Brownfield, Mailer, and Swift and Moore. The applicability of swept-frequency radar for testing purposes was investigated by Lundien, and his paper provides encouraging results. Klotz discusses a penetration test, the Windsor probe test, and correlates the measured penetration values with concrete strength results. Windsor probe results are also analyzed in the paper by Arni and are compared to corresponding results obtained by the Schmidt test hammer.

—Sandor Popovics

ADVANCES IN NONDESTRUCTIVE TESTING OF CONCRETE

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This paper reviews recent advances in the following nondestructive methods of testing concrete: ultrasonic, resonance, radioactive, electrical, initial surface absorption, chemical analysis, and hardness. A brief review is given of the research work in progress at the South Dakota School of Mines and Technology. There is a great potential in the concrete industry for nondestructive methods of testing concrete. Two major fields where those methods could prove to be superior to traditional methods are quality control in the construction of structural members, both precast and cast-in-place, and monitoring strength development to determine acceptable times for the removal of form work or transfer of prestressing force to concrete.

•THE PURPOSE of this paper is to review some of the recent advances in nondestructive testing methods as applied to concrete technology. Advances in both field and basic laboratory research methods are discussed. Special attention is given to advances and trends that, in the authors' opinions, will dominate the field in the future. An attempt is made to present a modern, balanced, realistic picture of nondestructive testing of concrete. References are used freely to indicate the sources from which the authors' opinions have been derived. References include those that present information on nondestructive testing of concrete (1, 2, 3, 4) and those that review the state of the art of nondestructive testing of concrete (5, 6, 7).

Nondestructive methods have been of considerable value in the testing of concrete. They are most useful in the testing of concrete structures in situ. In the laboratory, their special usefulness lies in the repetitive testing of the same specimen when the influence of time is evaluated. These are the two aspects of testing to which the nondestructive methods are better suited than the destructive methods.

It is, however, essential to keep the limitations of nondestructive testing methods in view and to exercise due caution in interpreting the results of these methods. All methods, including surface hardness methods, resonance methods, pulse velocity methods, and radioactive methods have been found to have their own limitations in application. Those methods cannot, at the present, replace the destructive test for the purposes of assessing the strength of concrete. Hence, in all cases, it is recommended that the data obtained be interpreted by taking into account all other available information under test, including the strength data from destructive testing, if any. Nondestructive methods of testing concrete may be used with greater confidence when the objective is only a comparative qualitative assessment or a change from a known quantitative state or standard.

Some of the recent trends in the studies concerning nondestructive methods appear to be determination of damping properties of concrete; resonance vibration of circular plates for determining the elastic properties of concrete (8); utilization of more than one observation in the statistical correlation of the strength of concrete (9), such as pulse velocity and damping constant (10) or these two along with rebound hammer observations (8); and a differentiation between the sharpness and flatness of the onset of the pulse. The use of more than one method to solve a particular problem is certainly logical in view of the inherent complexity of concrete. It is expected that this trend will continue.

The attempts are toward either developing new procedures of estimating the strength and other properties of concrete or using the existing methods for getting more reliable

and dependable information of the quality of concrete. Although every such attempt adds to the usefulness of nondestructive methods, investigations should also be directed toward development of altogether new methods and new instruments that can overcome many of the limitations of the existing methods.

"In all but the methods employing radioactive isotopes," states Jones (3), "the experimental techniques have now reached a stage when no major development is likely to occur for many years and, in most cases, commercial apparatus is readily available." It is, therefore, to be inferred that instrumentation of hardness, resonance, and pulse velocity methods do not constitute a potential field for a major breakthrough, and some other properties must be effectively used in the development of new nondestructive methods for testing concrete.

ULTRASONIC TESTING OF CONCRETE

The ultrasonic testing of concrete is mainly based on straightforward measurements of longitudinal pulse velocity; the pulse is produced by an electroacoustical transducer, which is held in contact with one surface of the concrete member under test and received by a similar transducer in contact with the other surface. There are several ways in which the behavior of ultrasonic pulses in concrete can be used to obtain useful information, particularly with regard to elastic stiffness and strength. Generally the elastic modulus of concrete increases with density and so does the velocity of ultrasonic pulses. It is for this reason that pulse velocity varies from point to point in a piece of concrete because density and elastic modulus are by no means constant throughout the concrete. However, between pulse velocity and static modulus there is a simple empirical correlation that holds good over quite a wide range of different concrete compositions (11). Both elastic modulus and pulse velocity are related to strength or quality of concrete.

Most of the experimental work carried out to evaluate the results of ultrasonic testing of concrete has been based on a comparison between pulse velocity and the standard crushing test. The statistical confidence of this correlation is not so high as concrete engineers would like. The reason is that the correlation is influenced by a number of factors such as cement content, aggregate type, age of concrete, and curing temperature. Those factors would have to be taken into account if a reasonable prediction of crushing strength has to be made from pulse velocity measurements.

Ultrasonic testing is also used for locating flaws, voids, or other defects in concrete (12); the technique is based on the negligible transmission of ultrasonic energy across a concrete-air interface. Ultrasonics is also used for defectoscopy investigations for location of weaknesses and damaged units and for analysis when failure has occurred. It is cheaper than drilling test cores (6). A combination of surface pulse propagation and longitudinal thickness resonance techniques (6) or pulse reflection has been developed to provide estimates of thickness of a concrete slab.

There are many applications of ultrasonic testing to field problems during the construction and acceptance phase and the evaluation of damaged concrete structures (2, 13). During the construction phase, ultrasonic testing can be used to determine consolidation and filling of deep forms. Voids may be detected in the filling of deep forms. Inadequate consolidation will result in reduced velocity readings. Detecting and correcting potential problems immediately in the field result in significant long-term economics.

Ultrasonic testing of concrete at an early age in the field can be used to determine setting characteristics and evaluate the action of accelerators and retarders. Furthermore, ultrasonic testing as early as 10 hours can provide an estimate of 7- and 28-day compressive strengths. Because the strength of concrete depends on many factors, all nondestructive methods for estimating it should be used with caution. Calibration data should be prepared for each particular mix and aggregate used. Care should be taken if any parameters having an effect on concrete strength change after the calibration is complete. Galan (10) states that "estimating concrete strength, especially in a water saturated condition, by pulse velocity alone is unreliable and in most cases impractical." Galan advocates measuring both the pulse velocity and the damping constant of the ultrasonic impulse in order to assess both elastic and inelastic properties of concrete.

Ultrasonic testing has been used successfully in prestressing and precasting operations where quality control is generally better than that in the field. The added variables of cast-in-place operations require more care on the part of the operator running the ultrasonic tests.

The pulse velocity of concrete may be used as an acceptance criterion in its own right. High-quality concrete has a pulse velocity markedly greater than that of poor-quality concrete.

In older and potentially damaged structures, the integrity of the concrete can be assessed by ultrasonic means. In this case, cores may be cut from representative parts of the structure to provide calibration data. Ultrasonic testing may also be used to predict accumulating damage under fatigue conditions as well as damage resulting from fires, earthquakes, and overloads.

Measurements of concrete pavement thickness may be made by using pulse velocity measurement in conjunction with resonance testing as discussed by Muenow (14). Other techniques for measuring pavement thickness by using the transmission of mechanical energy as a base are discussed by Howkins (4).

Assessment of Ultrasonic Testing of Reinforced Concrete Beams

A recent investigation (15) examined the validity of pulse velocity measurements in comparison with compressive strength tests when the structural quality of reinforced concrete beams is judged. It has been shown that pulse velocity measurements are more successful than crushing tests of companion specimens in assessing the flexural strength of beams. Although most of the investigation has been concerned with beam failure in bending, part of the program considered the prediction of shear failure from ultrasonic tests. It has also been shown that ultrasonic tests can clearly identify regions of low-strength concrete in reinforced concrete beams, and the influence of these weak zones can be predicted with reasonable accuracy. Evidence has been presented to show that the flexural stiffness of beams is closely related to ultrasonic pulse velocity in the concrete.

Pulse Velocity Technique as Applied to Small Concrete Specimens Under Repeated Compressive Loads

An ultrasonic pulse velocity technique has been applied by Raju (16) for the study of microcracks formed in high-strength concrete in a state of uniaxial compression under static and repeated loads. The tests were conducted on prismatic concrete specimens. They have revealed significant differences in the magnitude of pulse velocity decrease under static- and repeated-load systems. The progressive nature of the failure in concrete under repeated loads was studied by the parameter percentage decrease in pulse velocity in relation to the percentage of fatigue life. An empirical relation between the parameters was established from the test data, which could be used to predict the remaining fatigue life of a partially fatigued specimen.

The ultrasonic pulse velocity technique first introduced by Long, Kurtz, and Sandenaw (17) is essentially a nondestructive method of testing the quality of concrete by transmitting a vibrational pulse to travel a known distance through concrete. This technique has a definite advantage over the resonant frequency technique in that it could be used on any element irrespective of size and shape. This factor becomes important for testing the quality of concrete that forms part of a built-in structure.

Many investigators (18, 19) have used the longitudinal wave velocity method for detecting the formation of cracks inside concrete specimens; of these, Jones has conducted the most comprehensive study of microcracking under short-term static loads. The purpose of Raju's investigation was to extend the use of this well-established technique to repeated load situations, the data from which could be helpful in understanding the mechanism of fatigue failure in concrete.

The pulse velocity studies have demonstrated that sonic techniques can be advantageously used for detecting the development and growth of cracks under repeated loads, as shown by the progressive changes in the pulse velocity. The technique has a considerable advantage because of the ease of setup and use in the laboratory or field; and

the method permits an assessment of the condition of concrete along the entire path length, in contrast to the strain measurements that are made only on the surface. The use of pulse velocity and fatigue life relations determined from tests on small specimens, such as those used to predict the remaining fatigue life of larger structural members, should be made with caution. More tests are needed to study the effect of size on changes in pulse velocity in relation to fatigue life.

Significant differences were observed in the magnitude of decrease in pulse velocity between static and fatigue tests. The average decrease of pulse velocity in fatigue tests was found to be nearly 3 times that observed in static tests. The reasons for this considerable difference were investigated by conducting separate static and fatigue tests on similar concrete prisms with their surfaces ground so as to expose the aggregate matrix interface clearly. The crack data resulting from the microscopic studies during the tests were grouped into bond, matrix, and aggregate cracks. One study indicated that the major proportion in both static and fatigue tests was bond cracks, varying from 55 percent of the total length in static to 75 percent in fatigue tests. However, their orientation appears to have an almost identical distribution in each type of test with the greater part of their length nearly parallel to the direction of compressive stress.

The large magnitude of bond cracks accounts for the large decrease in the pulse velocity observed in fatigue tests. The constant stress amplitude used in fatigue tests favors the growth of a large number of local bond failures; in static tests only a few well-defined cracks form and grow. The large decreases in pulse velocity and modulus of elasticity of the specimens that fail under repeated loads indicate that the basic fracture mechanics concepts are qualitatively true for concrete in a state of uniaxial compressive stress.

Under repeated loads, the progressive nature of the failure provides excellent possibilities for using the pulse velocity technique with brittle materials. More research in the field of size effect in relation to changes in pulse velocity will help to make the ultrasonic pulse velocity method valuable in assessing the remaining fatigue life of a partially fatigued structural member in the field so that replacement or strengthening can be effected whenever necessary.

Raju has drawn the following conclusions from his investigation:

1. A significant decrease in the pulse velocity occurs in the lateral and longitudinal directions for concrete specimens subjected to repeated compressive loads of intensity 65 to 85 percent of the static ultimate strength;
2. The magnitude of decrease in the pulse velocity in the lateral direction under repeated compressive loads is nearly 3 times greater than that under static loads; and
3. The pulse velocity decreases with fatigue life at an increasing rate, and an empirical relation established between the parameters can be used to estimate the remaining fatigue life of a partially fatigued specimen.

Improvement in Instrumentation

One of the major difficulties in the popular use of nondestructive test methods, particularly the pulse velocity method, is the complicated and expensive instrumentation. However, new digital instruments that could revolutionize ultrasonic testing of concrete have been developed (20, 21) and are easy to operate, quick, cheap, and more accurate than currently available instruments.

The traditional equipment uses a cathode-ray tube to display the onset of the pulse and its position relative to timing marks superimposed on the same display. The use of this type of equipment requires patience and skill that are often difficult to maintain under arduous conditions usually prevalent on construction sites or in precast concrete factories. An instrument recently developed by Elvery and Vale (21) has no cathode-ray tube but gives a direct digital display of the time of transmission of an ultrasonic pulse passing from a transmitting to a receiving transducer; and it is compact (10.5 by 4.3 by 6.2 in.) and portable, demands less operator skill, and enables testing to be done more rapidly. This has a range of 1 to 1,000 μS and an accuracy of ±1 count.

RESONANCE TESTS

For the measurement of the elastic moduli of concrete, resonance techniques have become part of concrete testing technology. In this country and in many other countries, including Russia, India, Japan, Poland, and Czechoslovakia, this test has been included in the standards. Resonance methods are also specified in this country as a means of determining resistance to frost, and they have been used in other countries to determine resistance to attack by sulfate or sewage.

The main disadvantage of this method is that it is restricted to specimens of shapes for which it is possible to derive the frequency equations. Resonance techniques are rarely applicable to structural concrete for which ultrasonic methods are more suitable for in situ testing.

RADIOACTIVITY METHODS

Radioactivity methods are being increasingly used for the identification of defects that develop in concrete during construction and are hidden from the eye. When a beam of X-ray or gamma radiation passes through concrete, more radiation is absorbed by the denser parts than by the less dense parts of concrete. Gamma rays differ from X-rays only in their origin. The two main techniques used are radiography and radiometry. In both cases, gamma radiation is preferred because it is more portable than X-rays, is easier to use on sites, can penetrate thicker sections of concrete, and is cheaper than X-rays.

Radiography

Gamma radiography is now an established technique. The observation is made by the interaction of the emergent radiation with a suitable photographic film. Voids in the concrete will produce more intensity, and the steel in reinforced concrete will attenuate the radiation more than the concrete and consequently produce less interaction with the film. In a recent paper (22) Forrester reviewed the principles, techniques, and limitations of gamma radiography of concrete. The main use of radiography to date has been to detect voids in concrete and voids in the grouting of post-tensioned concrete, to locate steel in reinforced concrete, to determine the quality of mortar joints in precast prestressed units, to detect variable compaction in concrete of thickness up to 2 ft, and to measure the extent of corrosion. Gamma radiography is now being used increasingly in Great Britain where a British Standard Specification is being proposed for its use (22).

Radiometry

In radiometry, variations in the gamma intensity are detected by radiation detectors, such as Geiger or scintillation counters, and measured by associated electronic apparatus. There is an approximately linear relationship between attenuation of gamma radiation and density of concrete, and that provides a basis for measurement of the density of concrete.

Radiometry is used in Russia and East Germany for measuring density and its variation in precast concrete units (6). It is especially suitable for testing large numbers of precast units of a given shape and is used at the Road Research Laboratory in England for measuring density variations in cores cut from pavements (23). The core is rotated through a gamma-ray beam, and the count variation is detected by a rate meter. The change in count rate versus the depth in the core is automatically plotted on a chart record.

Instruments used for the in situ determination of density of concrete are either the backscattering type or the transmission type. The backscatter method is used to detect voids in concrete under steel plating where the use of ultrasonic reflection would cause confusion. The transmission type has been employed in the study of density variations with depth in columns and deep beams (24).

Microwave Absorption

Microwave absorption provides one way of measuring the moisture content in various materials including concrete. The method is based on the much higher absorption of microwaves by moisture than by the material in which the water is dispersed (7).

Radioisotope X-Ray Methods for Field Analysis of Wet Concrete Quality

One of the most outstanding problems today in concrete technology is that of quality control of concrete. Criteria of quality are based on the strength of the concrete, and currently that can only be measured after the concrete sets and cures. It may then be too late and very expensive to rectify mistakes. The strength of concrete is related to the cement and water contents, the bulk density, and the degree of air entrainment of the wet mix. Ideally, on-site measurement of these parameters should be made within the short time available between delivery and pouring. Unsatisfactory concrete could then be rejected on the spot and costly mistakes avoided.

Nucleonic density and moisture gages have been available for some time, and their application to concrete analysis and other construction materials is increasing every year. These instruments are capable of rapid nondestructive measurement and, if properly employed, give adequate accuracy of concrete bulk density and water content. Other non-nuclear methods are also available for measuring air entrainment and concrete consistency. No other suitably rapid method has yet been developed, however, for on-site measurement of the cement content of the mix, except for the pioneering work of Berry and Furuta (22) on a radioisotope X-ray methodology and conceptual instrument design for field analysis of wet concrete quality.

Their work has shown that the technique of low-energy gamma-ray scattering offers much promise for the analysis of cement in wet concrete mix. A practical arrangement of source and detector and a choice of radiation energy that afford good sensitivity while being insensitive to the coarse particle size distribution have been obtained. The measurement is relatively insensitive to moisture content per se but does depend on the sample bulk density and chemical nature of the aggregate. For a known aggregate, an accuracy of ±0.3 sack/cu yd is possible provided the density is determined to about ±1 lb/cu ft. A theoretical treatment of the problem has been shown to give a satisfactory explanation of the experimental observations. The theory also suggests that a practical self-contained instrument can be developed to yield the cement content independently of bulk density, water content, and aggregate type. To correct for the latter requires a sample of the raw aggregate for normalization purposes. The conceptual design of a portable instrument suitable for field assay was obtained.

Berry and Furuta recommend that future study should include the optimization of gage parameters, for example, the source-detector spacing and scattering geometry of the 2 radiation beams, because it is very important that both beams effectively see the same sample. Further experimental work on actual concrete mixes is also necessary to establish the effectiveness of the matrix correction technique and to develop a simple calibration procedure for variable aggregate type. It is envisaged that following a short period of proof testing in the laboratory the instrument will undergo further tests in the field by highway engineers.

COVER METERS

Position of Reinforcement

The depth of cover and condition of the steel can be determined by nondestructive test of in-place concrete. Gamma radiography is used to ascertain the position and condition of reinforcement in concrete. There are simpler magnetic devices for measuring the depth of the reinforcement near the surface. Cover meters, as they are called, are both portable and relatively cheap. One British cover meter (6) operates by measuring the reluctance of a magnetic circuit incorporated in a search unit; it consists of 2 coils mounted on a U-shaped core. An alternating current is passed through one of the coils, and the current induced in the other is measured; this depends on the mutual inductance of the coils and hence on the distance of the search unit from the reinforcement. A similar cover meter has also been developed in Holland.

Neutron Meter for Moisture Measurements

The use of a neutron meter is as feasible in moisture measurement in unsaturated and saturated zones of concrete as in soils (25). It is a nuclear technique using isotopes and radiation technology, primarily with the aid of commercial instruments. There is a slow but steady increase in its application, even though field calibration remains a rather difficult problem. A real need exists for research and development to increase the reliability of the meters in field use (26). The variation of moisture content with depth in columns and deep forms has been studied by using this method (24).

ELECTRICAL METHODS

Moisture Content

When the moisture content of concrete changes, its dielectrical properties are changed. By measuring these changes by electrical methods, one can determine the moisture content. An instrument that measures the changes of capacitance by using fixed electrode geometries is being used in France to measure the moisture of plaster walls to ascertain when they can be painted (6).

Electrical resistance probes have been used in tracing moisture permeation through concrete (28). Concrete specimens were subjected to a constant head of water while electrical resistance measurements were made periodically by means of embedded pairs of electrodes. The presence of moisture lowered the local resistance and consequently indicated the depth of penetration. Results showed that oven-dried specimens were fully permeated within a few days, but then, during a period of 120 days, hardening and reduction of permeability occurred.

Electrical Measurements of Corrosion of Reinforcing Steel

The extent of corrosion of reinforcing steel in bridges exposed to a variety of climatic conditions can be assessed by electrical measurements (29). The measuring technique consists of using a copper sulfate half-cell and voltmeter.

Laboratory results of the use of this equipment were previously reported by the California Division of Highways. That agency also reported on the results of using this equipment on bridges in California. The Federal Highway Administration has made electrical measurements of corrosion on reinforcing steel in bridge decks exposed to a number of different climatic conditions both in the South and as far north as South Dakota.

Pavement Thickness by Electrical Resistivity Measurements

Several methods of nondestructively measuring the thickness of concrete pavement are discussed by Howkins (4). One method not mentioned is the electrical resistivity method that is being studied at the South Dakota School of Mines and Technology. This method was originally proposed by Moore (30) and is an adaptation of the resistivity method commonly used in geophysical exploration. Resistivity measurements are relatively easy to make, and the equipment is relatively inexpensive. These factors both favor the use of electrical resistivity to determine pavement thickness. The primary difficulty lies in the interpretation of the field readings. Several methods of interpretation have been proposed for geophysical exploration and are currently being evaluated for use on concrete pavement. Results from field tests are encouraging; however, no report will be made until the limitations of the method are more clearly defined.

The electrical resistivity of concrete varies from that of an electrolyte before setting to that of a semiconductor after aging. It has been reported that the electrical resistivity of concrete varies exponentially with temperature. The higher the temperature is, the lower the resistivity is. Furthermore, the resistivity of concrete is highly dependent on the moisture content of the material. Fortunately, all of the resistivity methods for determining the thickness of concrete pavement depend only on the ratio of the resistivities of the layers involved in the pavement system.

The temperature and moisture dependence of the electrical resistivity does pose the problem of interpreting the data in the presence of temperature gradients or moisture gradients or both through the depth of the pavement. All resistivity methods assume that the resistivity of each layer is constant or varies only slightly with depth. These potential problems are now under consideration.

Cumulative Damage in Aggregates by Electrical Resistivity Measurements

The electrical resistivity of rocks has been used as an indicator of accumulating damage (31). In dry rock, it was hypothesized that, as damage accumulated in the microstructure, more cracks opened and the electrical resistivity increased. The experimental data tend to confirm this hypothesis. Other investigators have shown that the resistivity of saturated rock tends to decrease as failure is approached. Here the cracks that open apparently fill with water, which conducts better than the rock. Because concrete may be thought of as a synthetic rock, it is anticipated that the same behavior may be observed. A program to study the variation of electrical resistivity with accumulating damage in concrete is expected to begin presently.

INITIAL SURFACE ABSORPTION METHOD

Levitt (32) has discussed the apparatus and techniques used for the measurement of permeability and absorption, and he has also shown how one can interpret the results obtained in terms of resistance to frost and chemical attack, color change, and lime bloom efflorescence. As a nondestructive test, the method has value as an alternative to the present destructive absorption test as far as durability is concerned.

CHEMICAL ANALYTICAL METHODS

A chemical analytical method of rapid analysis of fresh concrete, in which the composition of a sample can be determined in 10 to 15 min, is reported by Brown and Kelly (33). It is claimed that it is possible to determine with adequate accuracy the cement content, the water content, and the content of chloride added deliberately or adventitiously. In addition, water-cement and aggregate-cement ratios and the dosage of calcium chloride on the weight of cement can be determined by calculation. A full description of the development work and of an extended on-site trial has been reported (34). The possibility of the use of chemical analytical methods as a means of predicting the strength development characteristics of portland cement and predicting the strength of hardened concrete is under investigation.

HARDNESS METHODS

In a recent paper (35), Kolek argued that it is not only possible but also desirable to take advantage of the potential offered by the hardness methods and thereby to improve the general quality control, which prevails in concrete construction today, at relatively little extra cost. There are basically 2 methods: the rebound method and the indentation method. Both are well known and are used internationally. However, both methods are of an empirical nature, and several precautions must be taken to obtain meaningful results. The methods give only a rough idea of the quality of concrete. Hardness methods in combination with other nondestructive methods have been used with success, and the accuracy of a strength prediction has been improved in such cases. The successful practical use of the combination of rebound index and pulse velocity measurements in Romania has been reported by Facaoaru (36).

ACOUSTIC-EMISSION OR STRESS-WAVE-EMISSION TECHNIQUES

For nearly a decade acoustic-emission or stress-wave-emission techniques have been used to study the rate of cracking and the presence and growth of fatigue cracks in metals. Recently Green (37) extended the use of those techniques to detect the failure and progress of failure in cylindrical concrete specimens and a model prestressed concrete pressure vessel. His findings have shown that variations in the stress-wave-

emission data can determine impending failure and previous structural loading extremes. He also found a limited correlation between stress-wave emission and modulus of concrete.

Low-level acoustic emission is the characteristic and irreversible sound emitted when a material is plastically deformed. These waves propagate from their source as elastic stress waves and are detected as minute perturbations by sensors positioned on the surface of the test specimens. The variations in the time of stress-wave arrival at each sensor location are used to locate the origin of the wave. Sound is also emitted when a crack grows. This is acoustic emission. The intensity of sound emitted by a crack is a function of its size. When the crack reaches a critical size, the material will fail. The detection technique involves periodic proofing to loads greater than working load while simultaneously monitoring for acoustic emission. The detected stress-wave emissions are amplified, selectively filtered, conditioned, and then channeled to either a magnetic tape recorder or a specially developed digital computer (or both) for recording and analysis.

Acoustic emission during successive loading can be employed to nondestructively evaluate prior loading levels and to locate the origin of cracking, the zone of maximum specimen deterioration, and the prestressing steel failures. The deviation from elastic response of the material can also be detected from the acoustic-emission data.

BASIC MATERIALS RESEARCH

Microstructure Studies

Nondestructive and semi-nondestructive methods are being used in studying the microstructure of concrete. These studies will eventually lead to a better understanding of the phenomenological macroscopic behavior and to improved concrete and better design criteria. Bhargava (24) reports that radiography, penetrant (colored) dyes, acoustic emission, ultrasonic pulse velocity, surface observation by optical microscope, and the photoelastic method have all been used in various studies of the microstructure of concrete. An understanding of the initiation and growth of cracks in concrete structures under load is essential to the understanding and control of fracture. The analysis of accumulating damage and fatigue life awaits more basic research on the microstructure of concrete.

POTENTIALITIES OF NONDESTRUCTIVE TESTING

There is a great potential in the concrete industry for nondestructive methods of testing concrete. Of all the nondestructive testing techniques available, the ultrasonic method is potentially the most useful for assessing concrete quality in engineering structures (11). Although it has been successfully used in the role of a troubleshooter when faulty workmanship has been discovered or is suspected in concrete construction, its full potentialities have hardly been exploited yet. In particular there are at least 2 major fields of use in which it could prove to be superior to traditional methods: quality control in the construction of structural members, both precast and cast-in-place, and monitoring strength development to determine acceptable times for the removal of formwork or the transfer of prestressing force to the concrete. In regard to the quality control, ultrasonic testing provides an opportunity to test the actual concrete in the structure, and test zones can be chosen for their critical importance in having to resist the highest stresses.

Progress is being made in the nondestructive methods of testing concrete both in the field and in basic materials research on the microstructure. Nuclear methods are showing great progress and are receiving increasingly wider acceptance. Nevertheless, one of the greatest problems in nondestructive testing of concrete is education. Most engineers working with concrete in the field have been trained in civil engineering and do not have the background to work with ultrasonic, electrical, and nuclear equipment. The universities have an obligation to provide engineers with the basic background in applied physics necessary for evaluating, selecting, and using this type of equipment and for comprehending its limitations.

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NONDESTRUCTIVE TESTING OF CONCRETE BY WAVE VELOCITY METHODS: A LABORATORY AND FIELD STUDY

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An investigation was conducted to determine whether the compressive strength and modulus of elasticity of concrete could be determined from the velocity of an elastic compression wave traveling through it. The elastic wave velocities were usually determined by the seismic method. Two comparisons between seismic velocities and elastic wave velocities determined by the soniscope method are presented. A correlation between the seismic velocity and compressive strength of slabs and beams constructed of con-concrete containing San Gabriel drainage aggregate was developed. The rate of change of wave velocity to compressive strength was undesirably small. This correlation is not necessarily valid for concrete made with aggregate from other sources. It was also concluded that seismic velocities may be influenced by the presence of reinforcing steel. It was observed that changes in temperature in the 2- to 60-C range have negligible effect on the seismic velocity of concrete.

•IF THE velocity of an elastic compression wave traveling through concrete can be related to the compressive strength, modulus of elasticity, or other physical properties of the concrete, then the concrete can be tested nondestructively. Elastic wave velocity measurements can be performed more rapidly than conventional tests and, therefore, are more economical.

Jones (3), Whitehurst (6), and many other researchers have employed the soniscope method of measuring the wave velocity of concrete. Phelps and Cantor (4) employed the seismic method to measure elastic wave velocity and successfully predicted the deterioration of asphalt-overlaid concrete bridge decks.

Seismic velocity tests, similar in principle to those performed for geophysical purposes, were performed on concrete to determine the elastic wave velocities. The seismic velocity apparatus is shown in Figure 1. Travel times were measured in microseconds because the traverse lengths were shorter than 1 m and the seismic velocities are of the order of 3,000 m/sec. A DynaMetric microseismic timer, model 217, the only commercially available microseismic timer, was used throughout this investigation. A length of adhesive copper tape was attached to each test slab at the point of impact. Striking the tape with a pendulum hammer closed a circuit to start the timer and created an elastic wave that traveled through the concrete. A phonograph-needle transducer at the first station received the wave and stopped the timer. Several travel times were observed until a repeatable reading was obtained. The transducer was moved to each successive station and the process repeated. The procedure for testing rough-finished slabs in the field differed: The transducer remained at the initial station; the field impactor replaced the pendulum hammer as the impact source; the field impactor was moved to successive stations; and the repeatability criteria were relaxed. The pendulum hammer and transducer were mounted for tests on vertical surfaces. Cylinders and cores were tested in a holder that isolated them from external vibrations (1).

With the soniscope, a vibrating-sending transducer transmits elastic compression waves to the receiving transducer. The travel time is directly proportional to the phase shift of the wave between the 2 transducers. The distance between the transducers is divided by the travel time to give the wave velocity. It is preferable that a soniscope be simple to operate and have flexible diaphragm transducers to make positive contact with the concrete. A James V-Scope was used in this investigation (1, 2).

Deterioration studies were not included in this investigation because it was programmed for 1 year (July 1966 through June 1967) and because the compressive strength of concrete usually increases with age.

TEST PROGRAM

This investigation consisted of (a) a laboratory phase performed in Arcadia, California, to develop procedures and hypotheses, and (b) field investigations, performed near Happy Camp, California, to test these findings.

A group of test structures was constructed from 12 concrete mixes. It consisted of 12 slabs 1.2 by 1.8 m, 6 columns 0.6 by 0.6 by 2.4 m, 2 beams 0.4 by 0.5 by 2.1 m, 1 arch of the same dimensions as the beam with 0.2-m maximum rise, and 7 flexure beams 0.15 by 0.15 by 0.6 m. Ten of the concrete mixes contained river-run aggregate from the San Gabriel drainage. One mix contained aggregate from the Tujunga drainage. The nominal specific gravity of these aggregates was 2.65. Crestlite lightweight manufactured aggregate was used as the coarse aggregate in 1 mix. The slumps of these mixes ranged from 40 to 180 mm. The air content varied from 1.8 to 6.4 percent. Number 3 reinforcing bars were placed 50 mm beneath the surface of 2 slabs. A No. 8 reinforcing bar was partially embedded in the top of 1 flexure beam; a semicurcular section of the bar remained exposed. A No. 8 reinforcing bar was placed 50 mm below the top surface of another flexure beam.

Seismic velocity tests were performed on all the test structures except the flexure beams at 7, 28, and 133 days. The compressive strength of both cylinders cured adjacent to the slabs and 100-mm diameter cores from the test structures were determined at the same ages.

The adhesive copper tape could not be expected to adhere well to rough or dusting concrete that would probably be encountered in the field if the points of impact were not ground. This effect was determined by performing 10 seismic velocity tests on horizontal surfaces of the test structures. The points of impact were ground with a grinding bit to simulate field conditions. The seismic velocity tests were repeated by using the ground points of impact.

The possible effects of temperature on the seismic velocity of concrete was determined by testing 189-day old flexure beams first at 20 C. The beams were brought to temperature equilibrium at 60, 2, and -18 C and retested.

Seismic velocity tests were performed with the objective of locating reinforcing steel by detecting the characteristic seismic velocity of steel, approximately 5,100 m/sec. These tests were performed directly above the embedded reinforcing steel and on the exposed surface of the half-embedded No. 8 reinforcing bar.

Static modulus of elasticity tests were performed on nine 133-day old cylinders representing different batches of concrete. These tests were performed according to Texas Highway Department Method 421-A, modified by using a Testlab CE 2760 compressometer and by stopping the test at a smaller load to avoid damaging the compressometer.

The elastic wave velocity of 1 slab, 6 columns, and 9 pairs of companion cylinders was measured with the V-Scope. The concrete tested with the V-Scope ranged in age from 200 to 256 days. Both transducers were placed on the top surface of the slab. The columns and cylinders were tested by through transmission, with the transducers placed opposite each other.

Six existing bridges and 1 bridge under construction, the Indian Mill Bridge, were tested by seismic velocity and conventional methods. The seismic velocities of the deck and abutment of the Indian Mill Bridge were measured at 7 and 28 days. The compressive strength of cylinders and cores representing the structure were determined at the same time. The ages of the existing bridges ranged from 8 to 46 months. Two existing bridges were constructed of concrete containing nominally 2.65 specific gravity aggregate. The other bridges were constructed of concrete containing nominally 2.80 specific gravity aggregate.

Two pairs of comparison tests were performed on a smooth concrete curb to determine whether seismic velocity tests performed with the field impactor are more accurate than seismic velocity tests in which the point of impact had been ground. The pendulum hammer was used as the impact source for 1 test of each pair, and the field impactor was used for the other.

Figure 1. Seismic velocity apparatus.

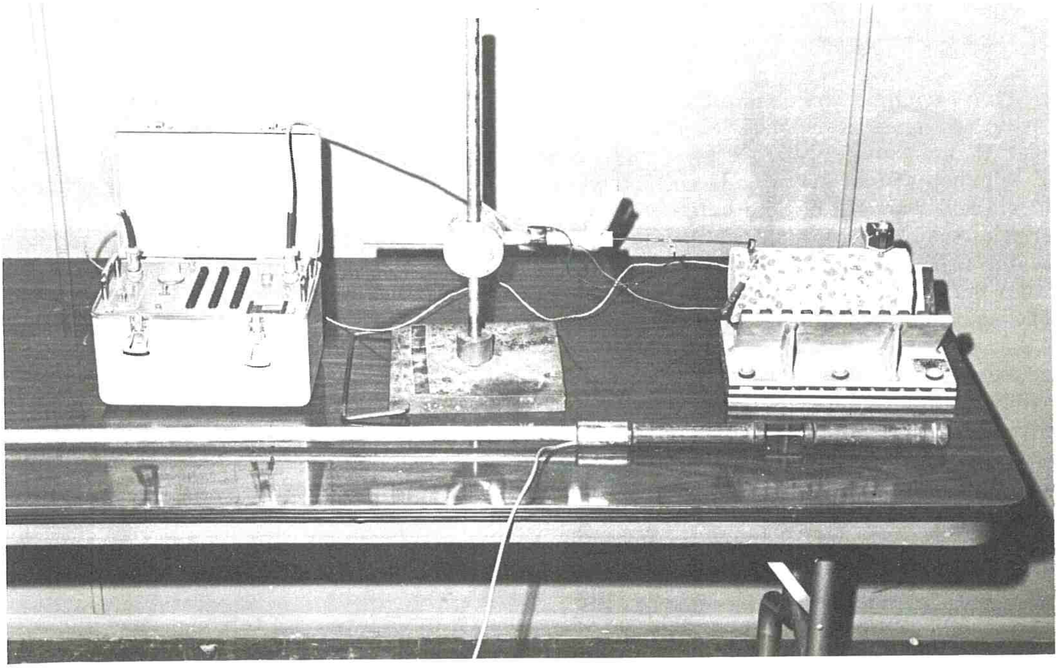
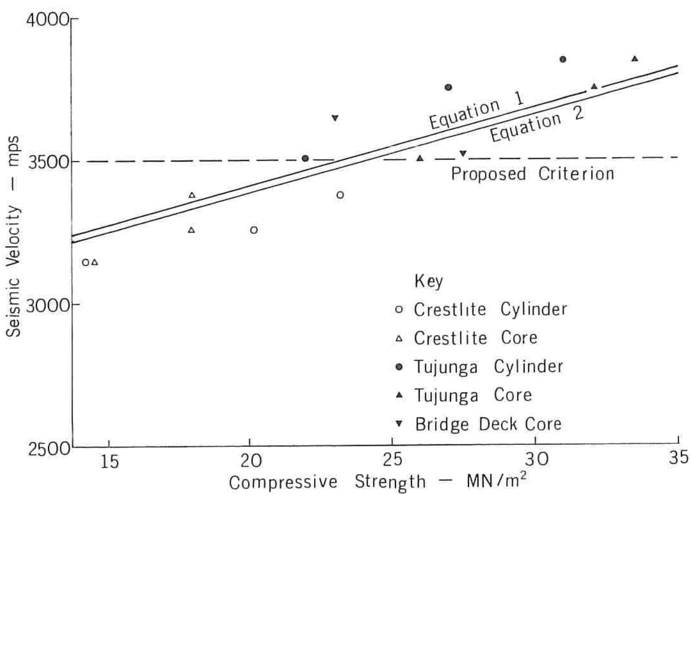


Figure 2. Seismic velocity and compressive strength relations.



RESULTS AND DISCUSSION

Horizontal Surfaces

Equations 1 and 2 were derived statistically from the results of seismic velocity and compressive strength measurements performed on the test slabs that had been constructed of San Gabriel drainage aggregate concrete.

For seismic velocity of slabs and compressive strength of cylinders,

$$SV = 3{,}050 + 22.6\,f \quad n = 30 \quad r = 0.87 \tag{1}$$

For seismic velocity of slabs and compressive strength of cores,

$$SV = 3{,}040 + 21.5\,f \quad n = 30 \quad r = 0.81 \tag{2}$$

where

SV = elastic wave velocity measured by the seismic technique, in meters per second;
r = coefficient of correlation;
n = number of data items (a data item may be the average of a maximum of 6 tests); and
f = compressive strength of concrete, in meganewtons per square meter.

Figure 2 shows the relations of seismic velocity and compressive strength of slabs constructed of concrete containing normal specific gravity aggregate and lightweight aggregate.

From Eqs. 1 and 2 and other analysis of the data from which they were derived, it is concluded that, if the seismic velocity of a slab equals or exceeds 3,500 m/sec, there is a 90 percent probability that the compressive strength will equal or exceed 20.7 MN/m^2. The 2 tests performed on bridge decks constructed of concrete containing nominally 2.65 specific gravity aggregate meet this proposed criterion. In the author's opinion, the rate of change of seismic velocity with compressive strength is undesirably low because variations in seismic velocity tests of 30 to 60 m/sec are commonplace.

Seismic velocity and compressive strength data from tests performed on bridge decks constructed from concrete containing nominally 2.80 specific gravity aggregate is represented by Eqs. 3 and 4.

For seismic velocity of Indian Mill Bridge deck and compressive strength of cylinders,

$$SV = 3{,}730 + 5.7\,f \quad n = 6 \quad r = 0.20 \tag{3}$$

For seismic velocity of bridge decks and compressive strength of cores,

$$SV = 3{,}710 + 6.3\,f \quad n = 10 \quad r = 0.15 \tag{4}$$

Only 25 percent of these results meet the 3,500 m/sec - 20.7 MN/m^2 criterion, probably because the concrete contains nominally 2.80 specific gravity aggregate. Because of the low correlation coefficient and rate of change of seismic velocity with compressive strength, neither Eq. 3 nor Eq. 4 can be used to establish a criterion for concrete containing nominally 2.80 specific gravity aggregate.

Both V-Scope transducers were placed on top of a test slab. The V-Scope velocity thus measured agreed exactly with the seismic velocity. The amplitude of the received signal was so small, however, that the author had no confidence in the result and thereafter used the V-Scope only for through transmission measurements.

Vertical Surfaces

Equations 5 and 6 were derived from the results of seismic velocity and compressive strength measurements performed on the test columns, beams, and arch.

For seismic velocity of vertical surfaces and compressive strength of cylinders,

$$SV = 2{,}960 + 15.1\ f \qquad n = 25 \qquad r = 0.34 \tag{5}$$

For seismic velocity of vertical surfaces and compressive strength of cores,

$$SV = 2{,}800 + 22.7\ f \qquad n = 25 \qquad r = 0.48 \tag{6}$$

Because of the poor seismic velocity and compressive strength correlation, no criterion was developed from the data represented by Eqs. 5 and 6. The seismic velocity of 6 of the 9 vertical surfaces did not increase with age as, in general, the slabs did. This leads the author to suspect that the seismic velocity tests performed on the vertical surfaces at 7, 28, and 133 days were not so reliable as the seismic velocity tests performed on the test slabs at the same age or on the columns at 200 to 256 days.

Seismic velocity and compressive strength data from tests performed on vertical surfaces of concrete containing nominally 2.80 specific gravity aggregate is represented by Eqs. 7 and 8.

For seismic velocity of vertical surfaces on Indian Mill Bridge deck and compressive strength of cylinders,

$$SV = 3{,}250 + 32.0\ f \qquad n = 6 \qquad r = 0.48 \tag{7}$$

For seismic velocity of vertical surfaces and compressive strength of cores,

$$SV = 3{,}500 + 23.8\ f \qquad n = 13 \qquad r = 0.51 \tag{8}$$

These seismic velocity compressive strength relationships differ significantly from those obtained during the laboratory investigations. The seismic velocity corresponding to a specific compressive strength is much higher in Eqs. 7 and 8 than in Eqs. 5 and 6, again probably because the specific gravity of the aggregate is higher.

The soniscope and seismic velocities of the columns were compared with the following result:

$$V = 1.07\ SV - 280 \qquad n = 6 \qquad r = 0.98 \tag{9}$$

where

V = elastic wave velocity measured by the soniscope technique, in meters per second.

The elastic wave velocities measured by these methods agree within 30 m/sec in the range of velocities measured. This agreement, in the author's opinion, substantiates the results of both tests performed at ages 200 to 256 days.

Cylinders and Cores

The seismic velocities of cylinders and cores are subject to error because of the short (less than 0.25 m) traverse length. Disturbance of the cores by high frequency vibrations during coring probably affected the seismic velocity or compressive strength of the cores or both (1). No meaningful correlation could be developed between the seismic velocity and compressive strength of individual cores and cylinders.

The soniscope and seismic velocities of companion cylinders having a 2.4 MN/m^2 maximum compressive strength variation between companion cylinders compared as follows:

For seismic velocity of first and second cylinders,

$$SV_2 = 870 + 0.72\ SV_1 \qquad n = 9 \qquad r = 0.57 \tag{10}$$

For soniscope velocity of first and second cylinders,

$$V_2 = 820 + 0.77\ V_1 \quad n = 9 \quad r = 0.91 \tag{11}$$

where subscripts refer to the first and second cylinder of each pair cast.

For comparison of seismic and soniscope velocities of individual cylinders,

$$SV = 67 + 0.87\ V \quad n = 18 \quad r = 0.70 \tag{12}$$

The V-Scope velocities of the pairs of companion cylinders agree with each other more closely than do the seismic velocities; the correlation coefficient of Eq. 11 is significantly higher than that of Eq. 10. The elastic wave velocity of cylinders, as measured by these 2 methods, differs significantly, discrediting one or both methods of measuring the elastic wave velocity through cylinders.

Miscellaneous Results

Dynamic elastic moduli were calculated from a relationship of the following form:

$$E_d = [(SV)^2\ (1 + \nu)\ \ (1 - 2\ \nu)]/(1 - \nu)$$

where

E_d = dynamic modulus of elasticity, in giganewtons per square meter;
d = density, in kilograms per cubic meter; and
ν = Poisson's ratio.

Poisson's ratio was assumed to be 0.2 (1, 5). These dynamic moduli are compared with static secant moduli between 89 kN and maximum test load as follows: for static and dynamic moduli of elasticity,

$$E_d = 3.86 + 1.24\ E_s \quad n = 9 \quad r = 0.86 \tag{13}$$

where

E_s = static modulus of elasticity, in giganewtons per square meter.

Because elastic moduli are important to structural engineers and a theoretical relationship exists between elastic wave velocity and elastic moduli, the author suggests that further research be conducted with the objective of determining elastic constants from seismic velocity measurements. He suggests that the specimens be at least 0.6 m in length to permit more accurate velocity determination and that shear wave velocities be measured so that Poisson's ratio can be calculated.

Only the characteristic seismic velocity of the concrete was detected when seismic velocity tests were performed directly above the No. 3 reinforcing bars embedded in the slabs. Two possible reasons why the steel was not located are (a) the seismic velocity of a long slender rod is significantly less than 5,100 m/sec and (b) energy losses reduced the amplitude of the elastic wave traveling through the steel to an undetectable magnitude.

When seismic velocity tests were performed on the exposed surface of the half-embedded No. 8 reinforcing bar, seismic velocities of more than 4,880 m/sec were measured. Seismic velocity tests performed directly above the No. 8 reinforcing bar embedded in the second beam resulted in a seismic velocity of 4,150 m/sec. This velocity, which is significantly higher than that of the concrete, is thought to be "seismic velocity influenced by steel." It may be significant as a source of error in testing reinforced structures and in locating steel.

Seismic velocity tests performed on flexure beams at 60 and 2 C resulted in changes in seismic velocity from that measured at 20 C of 60 m/sec or less in 10 of 11 tests. One 180-m/sec decrease in seismic velocity was measured at 60 C. In only 2 of 6 tests at -18 C were the changes in seismic velocity 60 m/sec or less. The maximum varia-

tion at -18 C was 490 m/sec. Ice films formed on the specimens tested at -18 C and interfered with the seismic velocity tests. It is concluded that for practical purposes the seismic velocity of concrete is unaffected by temperature changes in the 2 to 60 C range and that seismic velocity tests should not be performed below freezing because of the ice difficulty.

Five of the 10 seismic velocity tests performed after grinding the point of impact differed by 60 to 180 m/sec from the seismic velocity measured before grinding the point of impact. These variations ranged from 150 (higher) to 180 m/sec (lower).

The seismic velocity measured in 1 test using the field impactor as the impact source was identical to that measured in the test using the pendulum hammer. The seismic velocity measured in the test using the field impactor as the impact source was 90 m/sec higher than that measured in the test using the pendulum hammer in the second comparison. The field impactor was used as the impact source for testing broom-finished slabs during the field portion of this investigation because using the field impactor caused less seismic velocity variation than grinding the point of impact. Another advantage of using the field impactor on broom-finished slabs is that the transducer was positioned at the initial station for the entire test. Positive contact between the transducer and the concrete could be maintained more easily in this manner.

CONCLUSIONS AND SUGGESTIONS

It is difficult to predict the compressive strength of structural concrete by wave velocity methods in practice because (a) the wave velocity versus compressive strength relationships vary greatly depending on the aggregate and (b) the rate of change of wave velocity to compressive strength was found to be smaller than desirable in comparison to the precision of wave velocity measurements.

A relation between the seismic velocity and compressive strength of concrete slabs containing San Gabriel drainage aggregate was developed. If the seismic velocity of the slabs equals or exceeds 3,500 m/sec, there is a 90 percent probability that the compressive strength equals or exceeds 20.7 MN/m^2. Although 2 field tests performed on concrete containing nominally 2.65 specific gravity aggregate support the 3,500 m/sec - 20.7 MN/m^2 criterion, the validity range of this criterion has not been determined. This criterion definitely does not apply to concrete containing nominally 2.80 specific gravity aggregate and is not necessarily applicable to concrete containing aggregate other than the rounded San Gabriel drainage aggregate. The correlation between the seismic velocity of vertical surfaces and compressive strength developed in this investigation is inadequate for engineering purposes.

The author suggests that additional research in the following 2 areas would be worthwhile:

1. Investigate the seismic velocity-compressive strength relations of major pavement structures. Each structure should contain aggregate from a single geologic source. In this way the validity range of the 3,500 m/sec - 20.7 MN/m^2 criterion can be determined or a new criterion developed or both.
2. Determine elastic constants from elastic wave velocity measurements of specimens at least 0.6 m in length. A method of nondestructively measuring the actual modulus of elasticity of concrete is expected to be valuable to structural engineers.

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PAVEMENT THICKNESS MEASUREMENT USING ULTRASONIC TECHNIQUES

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The development and field evaluation of an ultrasonic pulse-echo type of instrument for measuring the thickness of portland cement concrete pavements are described. The results show that the system was accurate to within ±3 percent at 78 of the 100 test locations. Results were checked by core measurements made by the Ohio Department of Highways. The thickness gage was also evaluated on plastic concrete. The results show that this instrument is not capable of measuring the thickness of plastic concrete and also suggest that it is unlikely that any method using ultrasound could perform this function with the desired degree of accuracy. Laboratory experiments on blocks of bituminous concrete indicate that this instrument can be used to measure the thickness of this material at moderate temperatures.

•THE PAVEMENT THICKNESS gage described uses ultrasonic pulse techniques to measure the thickness of portland cement concrete, in place, in highways. Efforts to develop ultrasonic methods for accurate (±2 percent) measurement of portland cement concrete thicknesses have been generally unsuccessful in the United States and in other countries. Significant advances were reported by Jones (1), Bradfield (2), and Muenow (3), but in all cases it was found that accuracies better than ±5 percent were difficult, if not impossible, to obtain.

Two of the critical basic difficulties that have been encountered include (a) the relatively coarse-grained aggregates within the concrete that scatter, reflect, and severely attenuate high-frequency sound waves and (b) the surface roughness (particularly of bottom surfaces on gravel or other subbase structures) that tends to destroy coherent ultrasonic reflections from these surfaces.

The first phase of the development of the pavement thickness gage has already been reported (4). Since that time the interpretation of the acoustic signal spectrum has been revised, and a number of modifications, including the use of an independent acoustic velocity measuring procedure, have been made to the system. These changes have led to a more accurate and reliable system for measuring highway pavement thickness.

THEORY OF OPERATION

The pavement thickness gage measures pavement thickness by monitoring the time it takes for an ultrasonic pulse to travel down through the concrete and back again. The distance the sound travels is related to the transit time and sound velocity by the expression

$$d = 2.04\ T - Vt$$

where

- d = distance traveled, in.;
- T = pavement thickness or $d/2.04$, in.;
- V = velocity of sound propagation, in./sec; and
- t = transit time, sec.

A measure of the velocity with which the ultrasonic pulse travels through the concrete is obtained by measuring the time required by an ultrasonic pulse to travel between

2 transducers separated by a known distance on the surface of the pavement. The factor 2.04 is used rather than 2 because the ultrasonic beam does not follow a path straight down and back but rather follows a triangular path from the transmitter to the receiver via the bottom of the concrete pavement.

DESCRIPTION AND OPERATION

The thickness gage system consists of the following individual units: 1 large-area, ring-shaped transmitter, one 5-MHz receiver, two 4.25-in. diameter transducers, 1 type-0 Tektronix operational amplifier, 1 type-453 Tektronix oscilloscope, and specially designed pulse power supply. A photograph of the complete thickness gage is shown in Figure 1. The component parts and their functions are described in the following.

Transit-Time Measurement

The large-area transmitter, shown in detail in Figure 2, is used to obtain the transit-time measurement. This unit has an outside diameter of 18 in. and an inside diameter of 6 in. The ultrasonic generator is a mosaic piezoelectric radiator composed of 14 segments of a modified barium titanate material (Channelite 300). It has a characteristic thickness resonance of 200 kHz to which it responds when excited with an electrical impulse. The mosaic, although composed of several independent elements, responds acoustically as a single radiator when it is excited electrically. This capability has permitted high-energy ultrasonic pulses distributed over a broad area to be introduced into pavement materials for thickness-measuring purposes. The piezoelectric elements are glued to a 3-in. plastic buffer plate that supplies sufficient weight to the assembly to ensure good contact with the pavement.

The radiation pattern from the transmitter has an intensity peak about 20 in. from the unit. A receiver placed in the center of the transmitter would detect this peak when the unit was operating on a 10-in. thick pavement. A functional drawing of the transit-time measurement system is shown in Figure 3. This transmitter is designed for operation on 8- to 12-in. thick pavements. For pavements less than 8 in., a smaller diameter transmitter would give higher amplitude signals. Ideally the transmitter should be designed to have an intensity peak in the radiation pattern at a distance from the unit equal to twice the average thickness of the pavement on which the gage is used.

The signal reflected from the bottom of the pavement is detected by a 2.25-in. diameter, 5-MHz, lithium sulfate receiver. Earlier studies revealed that a receiver of the same frequency as the transmitter distorted the wave shape of the acoustic pulse and made signal identification difficult. The 5-MHz unit was found to be optimum for detecting the 200-kHz signal.

The electrical system used in the thickness gage is representative of a typical A-scan configuration often found in ultrasonic nondestructive testing. Thys type of presentation displays ultrasonic signal amplitude as a function of time. The unit providing the timing as well as the pulse excitation for the ultrasonic system is a typical multivibrator and thyratron pulser configuration. This unit is capable of delivering 600 volts at 75 amperes to a 5-ohm capacitative load for 1-μsec pulses.

In operation the transmitter and receiver are coupled to the pavement by a layer of glycerine. The receiver is moved about on the surface of the pavement within the center hole of the transmitter to optimize the signal response. The typical signal spectrum received is shown in Figure 4. The signal reflected from the bottom of the pavement is easily distinguished from the surface waves that have traveled from the transmitter to the receiver along the surface of the concrete. The transit time of the acoustic pulse through the concrete is obtained by subtracting the system delay time from the time measurement. The delay time, or the length of time necessary for the pulse to traverse the plastic block of the transmitter, is obtained by placing the receiver in direct contact with the base of the transmitter and recording the time of the signal detected.

Velocity Measurement

The transducers used to obtain the acoustic velocity are two 20-kHz rochelle salt units manufactured by James Electronics, Inc. The 4.25-in. diameter housings are

Figure 1. Complete pavement thickness gage system.

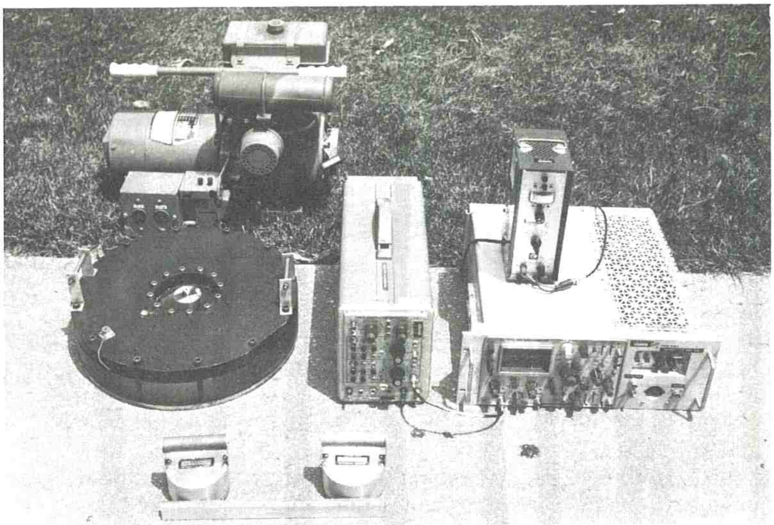
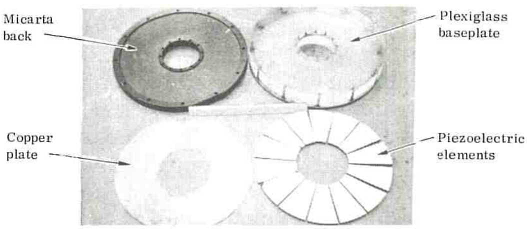


Figure 2. Disassembled large-area transmitter.



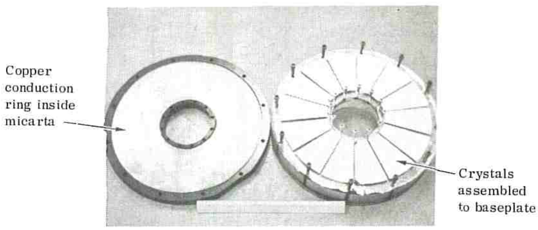


Figure 3. Functional drawing of the pavement thickness gage.

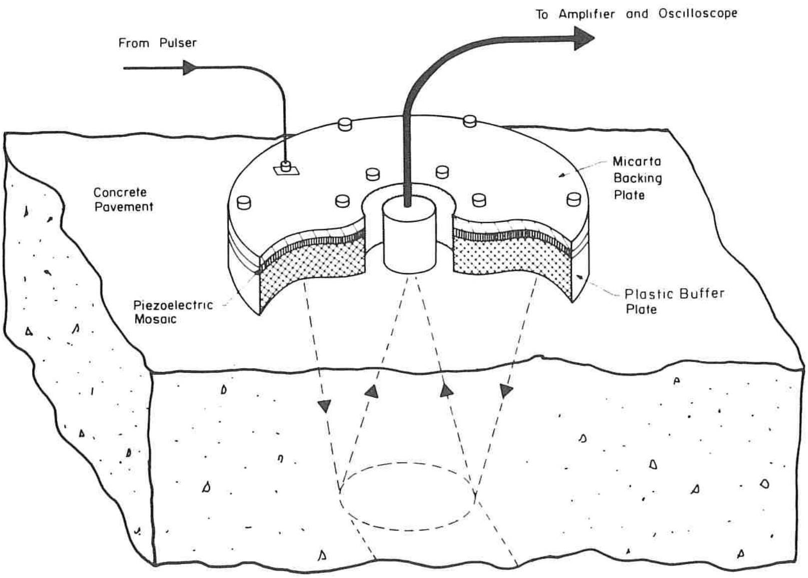
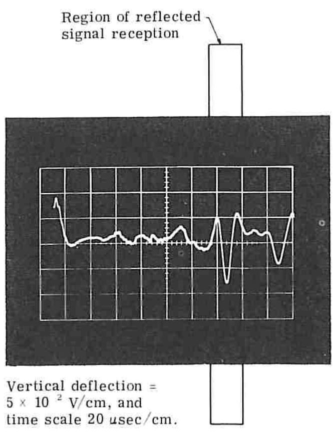


Figure 4. Response from tranmitter on 8-in. concrete.



filled with oil under pressure, thereby providing complete shock and vibration protection for the piezoelectric elements. The transducers are shown in the foreground in Figure 1. The pulse power supply and other electronic equipment is the same as those used for the transit-time measurement.

When the transducers have been placed a known distance on the surface of the pavement, one of the transducers is excited electrically and thereby transmits ultrasonic pulses into the concrete. The other transducer receives the pulses that are then displayed on the oscilloscope. The time of arrival of the pulses is recorded by noting the position of the first negative peak in the signal spectrum shown in Figure 5. The time required for the ultrasonic pulse to travel the known distance between the transducers is obtained by measuring the system delay time and subtracting it from the time measurement. This is obtained by holding the transducers in contact and noting the time of the signal displayed on the oscilloscope.

This technique samples the acoustic velocity in the concrete close to the surface of the pavement. If a velocity gradient exists through the thickness of the pavement, this method of measuring the acoustic velocity would be in error. However, laboratory experiments on test blocks, which permitted the placement of the receiver on the bottom of the concrete slabs, showed very good agreement with the surface method of obtaining the acoustic velocity. During these experiments the optimum separation between the transducers was found to be 18 in.

The complete thickness gage was evaluated under laboratory conditions on a number of test slabs in the 6- to 10-in. thickness range. The results showed that the gage was capable of consistently measuring the thickness of these blocks to within ±2 percent. The performance of the gage in actual highways is described in the following section. Improvements in the velocity measuring technique were made after the field tests and are described in a later section.

FIELD TEST RESULTS

Field tests were performed at each of the following locations in Ohio: Interstate 270, Columbus; Interstate 77, Cleveland; US-33, Athens; Ohio-48, Lebanon; and Interstate 90, Cleveland.

Approximately 100 core locations were examined. Fifty percent of all cores measured had accuracies of ±2 percent or better; 78 percent had errors less than ±3 percent. Greater accuracy would have been achieved if one core had been drilled from each section of highway to calibrate the thickness gage. Seventy-eight percent of the measurements would have had errors less than ±2 percent, and 94 percent would have had errors less than ±3 percent. The results of all the field tests are given in Table 1 and show the average thickness of the highway at each major test location as measured by both the acoustic and mechanical gages. The standard deviations of the acoustic measurements from the mechanical measurements are also given. The basic parameters of the highway, which could affect the quality and interpretation of the acoustic signal spectrum, are given in Table 2 for each test location.

Core measurements taken by a mechanical gage were provided by the testing laboratory of the Ohio Department of Highways. The results are shown in Figures 6 through 10 in a form that permits a clear comparison of the measurements made by the acoustic and mechanical gages at each core location. These plots show the difference between the acoustic and mechanical measurements. The results were also plotted with the use of a core calibration whenever this procedure significantly improved the accuracy of the results.

IMPROVED VELOCITY MEASUREMENT PROCEDURE

Transducer Orientation

Laboratory tests conducted after the field tests on the actual cored samples indicated that errors of as much as ±2 percent were encountered in the velocity measurements. Tests were performed to find the source of those errors. It was learned that, although the transducer housing is circular with a 4-in. diameter, the piezoelectric element is

Table 1. Summary of field test results.

Location	Number of Sites	Core Average (in.)	Gage Average (in.)	Standard Deviations (in.)
I-270	25	8.60	8.45	0.33
I-77	18	10.45	10.50	0.20
US-33	18	8.40	8.75	0.17
Ohio-48	20	9.30	9.20	0.26
I-90	18	10.70	10.35	0.13

Figure 5. Typical signal pattern received and used to measure local acoustic velocity.

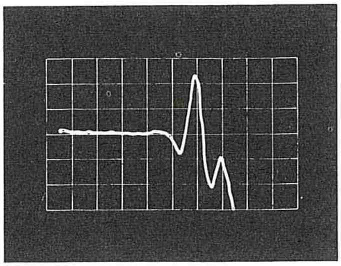


Table 2. Basic parameters at each test location.

Location	Aggregate Type	Top Surface Roughness	Subbase	Reinforcement
I-270	Gravel	Rough	Bituminous aggregate	0.725-in. longitudinal bars 6 in. apart and 0.5-in. transverse bars 34 in. apart
I-77	Slag	Smooth	Slag	Paving mesh
I-90	Slag	Smooth	Slag	Paving mesh
Ohio-48	Gravel	Smooth	Sand and gravel	Paving mesh
US-33	Gravel	Smooth	Sand and gravel	Paving mesh

Note: Paving mesh consists of 0.262-in. longitudinal bars 6 in. apart and 0.225-in. transverse bars 12 in. apart.

Figure 6. Field test results from I-270.

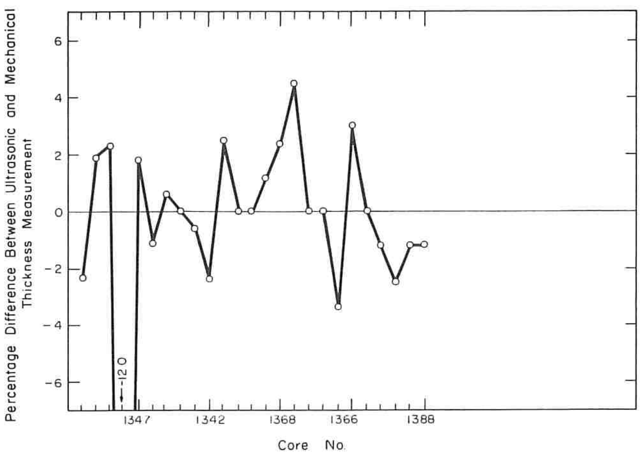


Figure 7. Field test results from I-77.

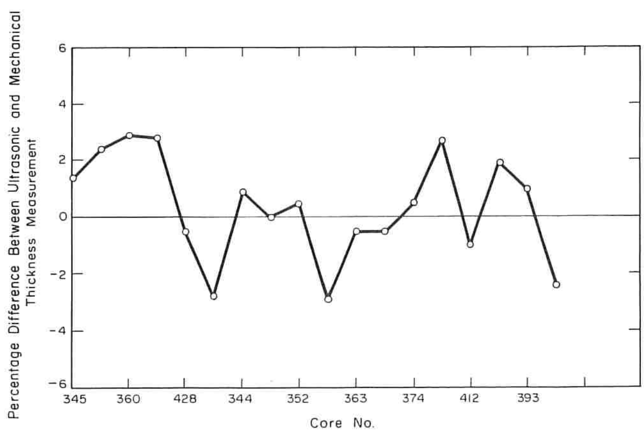


Figure 8. Field test results from US-33.

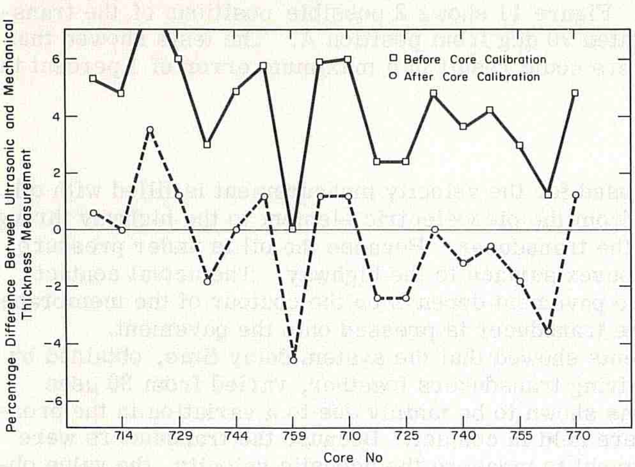


Figure 9. Field test results from Ohio-48.

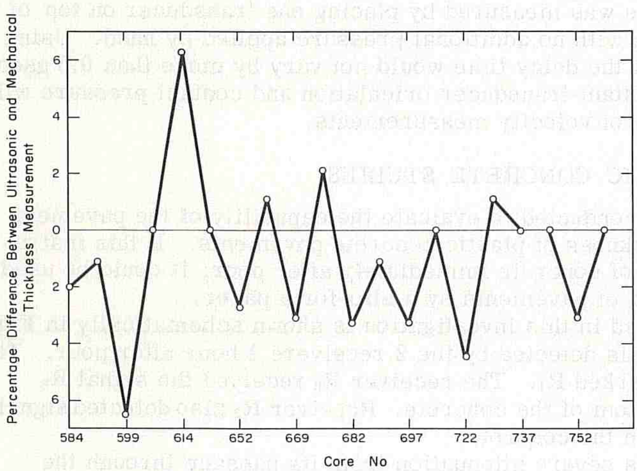
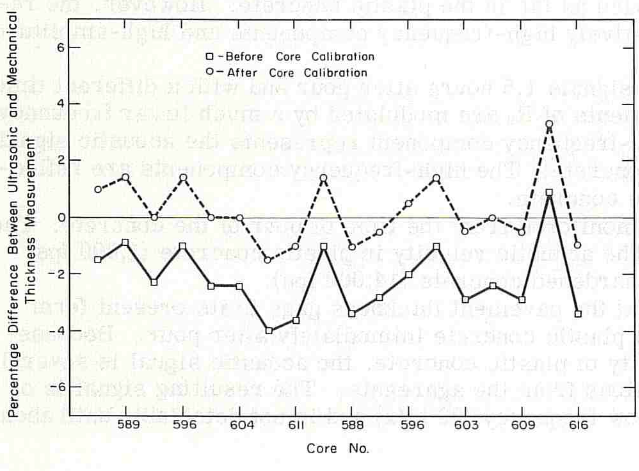


Figure 10. Field test results from I-90.



rectangular. The actual separation distance of the crystals, therefore, varies with the orientation of the transducers. Figure 11 shows 2 possible positions of the transducers, position B having been rotated 90 deg from position A. The tests showed that random orientation of the transducers could result in a maximum error of 1 percent in the velocity measurement.

Transducer Contact Pressure

The housing of the transducers used for the velocity measurement is filled with oil; the oil couples the acoustic output from the piezoelectric element to the highway through a rubber membrane at the base of the transducer. Because the oil is under pressure, the rubber membrane presents a convex surface to the highway. The actual contact area between the transducer and the pavement depends on the contour of the membrane surface and the force with which the transducer is pressed onto the pavement.

Analysis of the field measurements showed that the system delay time, obtained by pressing the transmitters and receiving transducers together, varied from 30 μsec (30×10^{-6} sec) to 35 μsec. This was shown to be mainly due to a variation in the pressure with which the transducers were held in contact. Because the transducers were also pressed by hand onto the pavement to measure the acoustic velocity, the value obtained was also subject to error due to pressure variations.

This problem was corrected by attaching 12.5-lb weights to each transducer. The oil pressure in the transducers was adjusted to achieve a 2-in. diameter contact area with the pavement. The delay time was measured by placing one transducer on top of the other and by taking the reading with no additional pressure applied by hand. Using this constant pressure showed that the delay time would not vary by more than 0.5 μsec. It is expected that the use of a constant transducer orientation and contact pressure will significantly improve the accuracy of velocity measurements.

PLASTIC CONCRETE STUDIES

A laboratory investigation was conducted to evaluate the capability of the pavement thickness gage to measure the thickness of plastic concrete pavements. If this instrument could measure the thickness of concrete immediately after pour, it could be used to monitor, and control, the laying of pavements by a slip-form paver.

The laboratory arrangement used in this investigation is shown schematically in Figure 12. Figure 13 shows the signals detected by the 2 receivers 1 hour after pour. The receiver R_1 received the signal marked R_1. The receiver R_2 received the signal R_2 after it was reflected from the bottom of the concrete. Receiver R_2 also detected signals reflected from the aggregate within the concrete.

The transmitted signal R_1 shows severe attenuation from its passage through the plastic concrete. The signal is also of very low frequency suggesting that only the lower frequency components of the acoustic pulse succeeded in penetrating the 4-in. plastic concrete. The reflected signal R_2 was expected to be of lower amplitude and frequency than R_1 because R_2 had traveled twice as far in the plastic concrete. However, the reflected signal R_2 contains comparatively high-frequency components and high-amplitude signal strength.

Figure 13 also shows the same signals 1.5 hours after pour and with a different time scale. The high-frequency components of R_2 are modulated by a much lower frequency wave form similar to R_1. The low-frequency component represents the acoustic signal reflected from the bottom of the concrete. The high-frequency components are reflections from the aggregate within the concrete.

The acoustic velocity was also monitored from the time of pour of the concrete. The results are shown in Figure 14. The acoustic velocity in plastic concrete (1,000 fps) is very low compared with that in hardened concrete (14,000 fps).

It was, therefore, concluded that the pavement thickness gage in its present form could not measure the thickness of plastic concrete immediately after pour. Because of the high degree of nonhomogeneity of plastic concrete, the acoustic signal is severely scattered and attenuated by reflections from the aggregate. The resulting signal is of comparatively low amplitude and low frequency (23 kHz) and is not detectable until about

Figure 11. The transducers in positions A and B, the 2 positions used during field tests.

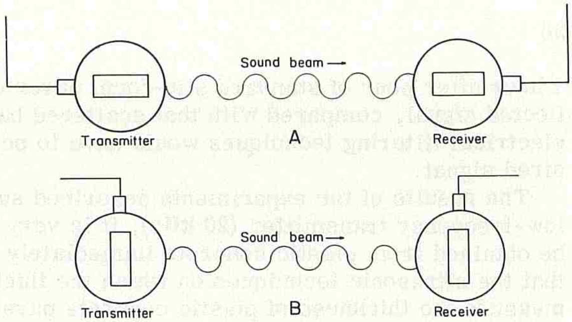


Figure 12. Experimental arrangement used to evaluate the capability of gage to measure plastic concrete.

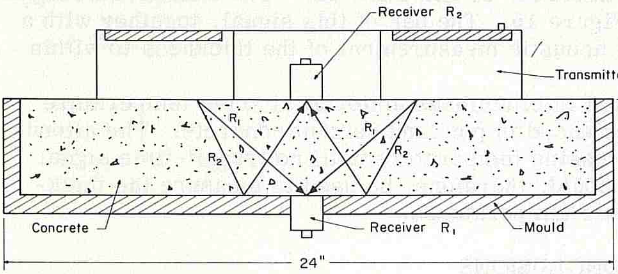
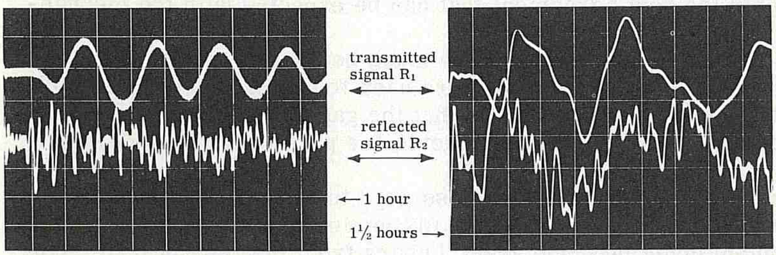


Figure 13. Transmitted and reflected signals through 4-in. plastic concrete 1 hour and 1½ hours after pouring.



Vertical deflection = 2×10^{-1} V/cm, and time scale = 0.2 μsec/cm.

Vertical deflection = 2×10^{-1} V/cm, and time scale = 0.1 μsec/cm.

Figure 14. Variation in acoustic velocity in concrete with 50 percent aggregate from time of pouring.

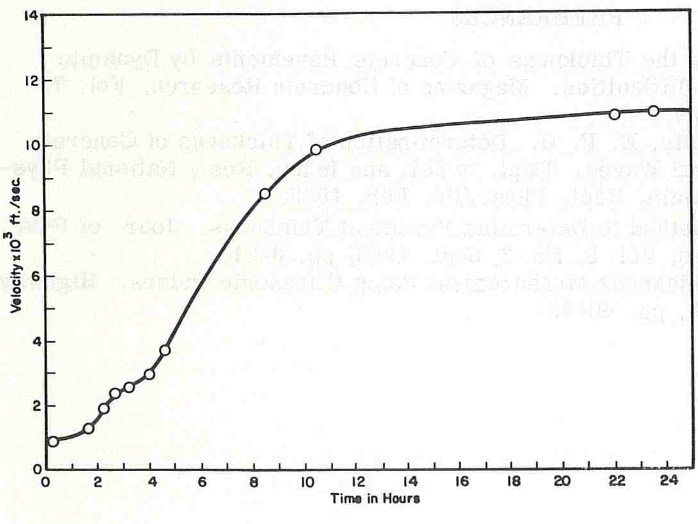
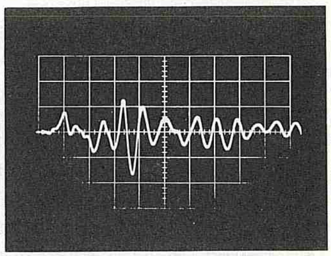


Figure 15. Typical transit-time signal recieved from 4-in. block of bituminous concrete.



Vertical deflection = 50 MV/cm, and time scale = 20 μsec/cm.

1 hour after pour of standard slip-form paver concrete. The low amplitude of the reflected signal, compared with that scattered back from the aggregate, suggests that electrical filtering techniques would have to be used to enable identification of the desired signal.

The results of the experiments described suggest that, even with a large area and low-frequency transmitter (20 kHz), it is very doubtful whether a usable signal could be obtained from plastic concrete immediately after pour. It is, therefore, concluded that the ultrasonic techniques on which the thickness gage is based cannot be used to measure the thickness of plastic concrete pavements.

THICKNESS MEASUREMENT OF BITUMINOUS CONCRETE

Tests were conducted on a 4-in. thick block of bituminous concrete to evaluate the capability of the gage to measure the thickness of this material. The transit-time signal received from the block is shown in Figure 15. The use of this signal, together with a velocity measurement, resulted in an acoustic measurement of the thickness to within 1 percent of the actual thickness.

The attenuation of the acoustic signal in bituminous material at room temperature (22 C) was found to be twice that experienced in portland cement concrete. The attenuation increased considerably with increasing temperature until no transit-time signal could be observed at 52 C. The gage could, therefore, be used to measure the thickness of bituminous concrete at moderate temperatures.

CONCLUSIONS

The pavement thickness gage has demonstrated its ability to measure the thickness of portland cement concrete pavements. Accuracies of ±3 percent at more than 90 percent of the test locations are expected with the use of the improved velocity measuring technique. That is probably the best agreement that can be expected with the mechanical measurement taken from cores.

It is expected that the ultrasonic gage would show much better agreement with the mechanical measurements if statistical methods were used to evaluate pavement thickness. Laboratory results have also demonstrated that the gage can measure the thickness of bituminous concrete provided the measurements are performed at moderate temperatures.

Tests on plastic concrete have shown the thickness gage to be incapable of measuring concrete immediately after pour. Depending on the properties of the mix, it is possible that a thickness measurement could be made about 3 hours from the time of pour, but by this time the concrete is generally in the transition region between the plastic and hardened states.

Additional field tests of the gage are now being performed in Ohio and in Pennsylvania.

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INVESTIGATION OF APPLICABILITY OF ACOUSTIC PULSE VELOCITY MEASUREMENTS TO EVALUATION OF QUALITY OF CONCRETE IN BRIDGE DECKS

Gilbert Swift and William M. Moore, Texas Transportation Institute, Texas A&M University

The compressional wave velocity and several other significant properties of concrete specimens were determined in the laboratory by using a wide variety of concrete compositions. It was found that the measured velocity and the unit weight could be used to estimate the elastic modulus of the concrete as measured either by stress-strain observations or by resonant frequency testing. It was also found that the compressional wave velocity in concretes of similar composition was generally indicative of quality. A portable velocity-measuring instrument employing some novel features was developed to measure acoustic pulse velocities on the accessible upper surface of bridge decks. It appears that this instrument fulfills its design objectives and is applicable to the problem of detecting the extent of deterioration of concrete in bridge decks.

•THIS PAPER presents some results of the "diagnosis" phase of a research study on the diagnosis, treatment, and repair of reinforced concrete bridge deck deterioration. The specific objective of this phase was to develop methods to evaluate the extent of deterioration in bridge decks.

Two defects have been considered to be of paramount importance: (a) delamination or the separation of the original slab into 2 or more approximately horizontal layers and (b) poor-quality concrete. An earlier paper (11) described the delamination portion of the research, and this present paper describes the work directed toward the detection of poor-quality or deteriorated concrete.

REVIEW OF LITERATURE AND SELECTION OF METHOD

According to the literature, acoustic pulse velocity measurements appeared to offer the most promising method for determining the quality of the concrete in bridge decks. As a first step, therefore, the relationship of acoustic wave velocity to other properties of concrete was explored. The literature indicates the existence of a general relation between concrete quality and acoustic velocity (1, 2, 3) and a theoretically based relationship between velocity, density, and elastic constants of concrete (3, 4, 5, 6, 7, 8). A relation has also been found between velocity and presence of certain cracks that lengthen the path and thus lower the observed velocity (1, 2, 3, 8).

Krautkramer and Krautkramer (1) have indicated the following relation between velocity and the quality of concrete:

Velocity (fps)	Quality of Concrete
15,100 and above	Very good
11,800 to 15,100	Good
9,850 to 11,800	Questionable to moderate
6,900 to 9,850	Bad
6,900 and below	Very bad

Breuning and Roggeveen (2) give somewhat similar data. Although these approximate relations neglect many factors that can influence the results, they imply that the extent of deterioration of concrete in a bridge deck might be determined by surveying the deck with an instrument that measures the acoustic wave velocity.

The theoretical relation among the compressional wave velocity, the density, and the elastic constants of a homogeneous elastic material is as follows:

$$V_c = \sqrt{(E/\rho)\{(1 - \mu)/[(1 + \mu)(1 - 2\mu)]\}} \tag{1}$$

where

V_c = compressional wave velocity;
E = Young's modulus of elasticity;
μ = Poisson's ratio;
ρ = mass density or W/g;
W = unit weight; and
g = acceleration due to gravity.

Although concrete is neither perfectly elastic nor homogeneous, the relation given in Eq. 1 has been reported by several investigators to be generally applicable to concrete (4, 5, 6). From it a value for Young's modulus can be computed, given the compressional wa"e velocity, the unit weight, and the value of Poisson's ratio.

$$E = (V_c^2W/g)\{[(1 + u)(1 - 2\mu)]/(1 - \mu)\} \tag{2}$$

This relation is such that, in the vicinity of Poisson's ratio equal to 0.20 (the vicinity applicable to both good and inferior concretes), small changes in Poisson's ratio have very little effect on the value obtained for the elastic modulus. For example, a 25 percent increase in Poisson's ratio changes the modulus by less than 7.5 percent. A decrease has less effect. Changes in unit weight have a directly proportional effect on the modulus value. Accordingly, one might expect to find that measurements of the compressional wave velocity could be used to compute a reasonably accurate value for the elastic modulus by using estimated values for Poisson's ratio and unit weight. To the degree that a decrease in the derived modulus value is an indication of deterioration, the relation given in Eq. 2 also implies that the extent of deterioration of concrete in a bridge deck might be determined by surveying the deck with a suitable compressional wave velocity measuring instrument.

The literature contains seemingly conflicting opinions as to the validity or the general applicability of those relations between the pulse velocity and other attributes of concrete. The apllication of pulse transmission is not recommended by ASTM for determination of strength or modulus (ASTM C 597). Manke and Gallaway (4) state that there appears to be some doubt as to the value of dynamic moduli calculated from measured pulse wave velocities. However, Manke and Gallaway also quote Whitehurst (3) who states, in effect, that the pulse velocity itself is as good a criterion for comparison of concretes as any other property that might be calculated from it.

A highly informative survey by Jones and Facaoaru (9) shows widespread use of the pulse velocity technique for estimating in situ strength of concrete. However, considerable divergence of opinion and practice was revealed by answers to the questions, What analytical formula is used? and What properties of the concrete should be varied in order to derive the correlation between pulse velocity and compressive strength of laboratory samples? A more positive finding is reported by Elvery and Din (10), who conclude that ultrasonic pulse testing provides a better correlation with beam strength than that given by control specimens.

In view of this range of opinion it was considered desirable to explore the underlying relations between acoustic pulse velocities and other properties of concrete so that they could be applied to the problem of detecting deterioration of concrete in bridge decks. Specifically, relations among the following variables were explored for a wide variety of concrete compositions: compressional wave velocity, dynamic modulus (from resonant frequency tests), elastic modulus (from stress-strain observations), unit weight, and compressive strength.

MEASUREMENT OF COMPRESSIONAL WAVE VELOCITY IN CONCRETE

Basically the compressional wave velocity of any material can be determined by initiating an acoustic impulse in the material and timing its travel over a known distance. In elastic solids several types of waves are generally produced in addition to the compressional wave. Fortunately, however, the compressional wave travels faster than the others. Hence, the first arrival of energy at a point not too distant from the source may be identified safely as being due to the compressional wave.

The ability to time the arrival of a wave accurately is a function of the abruptness or rise time of the wave. Accordingly, the acoustic impulse and the received wave should rise as steeply as possible. Most sonic and ultrasonic transducers are inherently resonant devices that produce and receive impulses having the form of lightly damped wave trains that oscillate numerous times while building up to their maximum amplitude. With such transducers the attainable timing accuracy is generally proportional to the frequency of the wave train. Therefore, for accurate velocity measurements, it is desirable to use as high a frequency as possible and to time the travel of the wave over as large a distance as possible. However, the properties of concrete prohibit the use of the high frequencies that are normally employed for testing metals and other relatively homogeneous elastic substances. The granular nature of concrete causes scattering and attenuation of waves whose length is comparable to, or shorter than, the size of the coarse aggregate particles. These effects set a limit somewhat below 100 kHz for the highest frequency that can be employed satisfactorily in concrete and a limit of a few feet to the practical distance range. Frequencies between 20 and kHz are, therefore, generally chosen for use with concrete.

Compressional wave velocities in various concretes ordinarily range between 8,000 and 16,000 fps. The waves thus traverse a distance of 1 ft in about 60 to 120 μsec. To determine their velocity with a precision in the order of 1 percent, one must, therefore, be able to define the onset of the wave train to within 0.6 to 1.2 μsec/ft of path length in the specimen. Such accuracy is not readily achieved with wave trains, whose oscillations each occupy 20 to 50 μsec and whose first oscillation is substantially smaller than the succeeding ones. Judgment of the observer is, thus, a highly significant factor in the timing process.

LABORATORY INSTRUMENTATION AND TECHNIQUE

The apparatus used for the laboratory measurements of compressional wave velocities is substantially the same as that described by Manke and Gallaway (4). It consists of a pulse generator, a commercially available oscilloscope equipped with a calibrated delayed sweep, and a pair of piezoelectric transducers. The assembled transducers are resonant at approximately 40 kHz. A repetition rate of 60 impulses per second is employed to facilitate observation.

The simplest measuring technique, as shown in Figure 1, may be described as "timing through" the specimen. Velocity is determined by observing the time interval between the occurrence of the driving impulse and the onset of the wave train received through the specimen and then subtracting the time interval observed with the transducers coupled directly together in the absence of the specimen. In taking this difference, one assumes that no change of the time delay within the transducers or in the acoustic couplings occurs when the specimen is introduced or removed. Accordingly, for reliable measurements it is essential to ensure good coupling. This is generally attained by applying a film of grease or starch that acts as a couplant between the transducers and the specimen and by applying a substantial pressure during the measurement.

An alternative technique that is applicable to specimens that have only one accessible flat side may be described as "timing along" the specimen. As shown in Figure 2, this technique utilizes a series of 2 or more observations made with various distances between the transducers. Velocity is obtained from the slope of the distance versus time plot of these observations. The interesection of this plot with the time axis is a measure of the time delay in the transducers and their couplings plus any time required by changes in the direction of the wave path. In this technique coupling delay remains an important factor affecting the accuracy of each individual observation, but its effect

Figure 1. Concrete specimen during measurement of acoustic pulse velocity by timing-through technique.

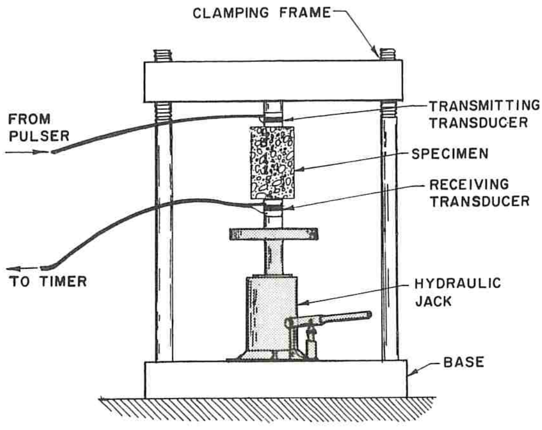


Figure 2. Concrete specimen during measurement of acoustic pulse velocity by timing-along technique.

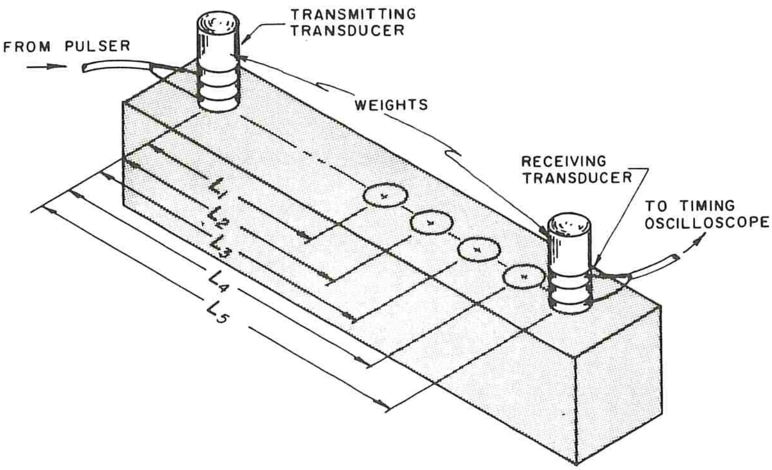
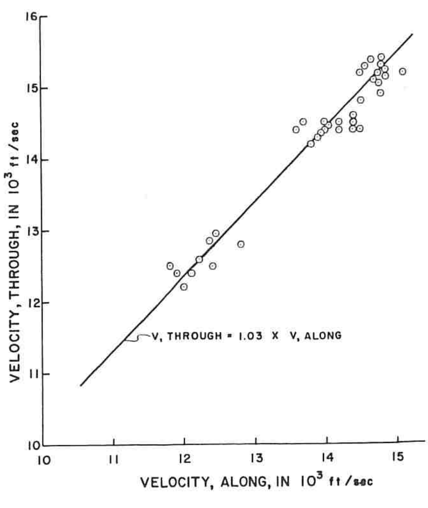


Figure 3. Comparison of velocities measured by using timing-along and timing-through techniques.



tends to average out among the series of observations. Adequate contact is obtained by placing a weight of a few pounds on each transducer and by using a suitable coupling agent.

LABORATORY RESULTS

Comparison of Timing-Along and Timing-Through Techniques

Because of its accessibility, the upper surface of bridge decks is a convenient place to take pulse velocity measurements with the timing-along technique. Accordingly, before an apparatus for such measurements on bridge decks was developed, a series of laboratory tests was made to examine the validity of this technique.

Thirty-six beams, 3 by 3 by 12 in., were made during this study from 12 batches of concrete (3 replicate, i.e., as nearly identical as possible beams from each batch) and were measured by both the timing-through and the timing-along methods. The batches contained 3 different types of aggregate and had widely varying cement factors. The results of the measurements made by the 2 methods are shown in Figure 3 and indicate a satisfactory agreement between the 2 measurement techniques. Thus, either of the 2 techniques may be applied for comparison of various concretes and for estimation of their velocity-related properties. A small bias was noted in the comparison in that the velocities obtained by timing through a given specimen averaged about 3 percent greater than the velocities obtained by timing along the same specimen. The reasons for this bias have not been determined.

Relation of Velocity to Dynamic Modulus

The theoretically based relation of velocity to the elastic modulus and the unit weight (Eq. 2) was examined through measurements made on 57 beams, 3 by 3 by 12 in. Thirty-six of these beams were the new beams mentioned earlier, and the remaining 21 were similar beams that had different unknown compositions and had been deteriorated by freeze-thaw cycling in a research study conducted several years previously. Velocities were determined by the timing-through technique, unit weights were determined from the weight and dimensions of the specimens, and the dynamic elastic moduli, E_r, were determined by the transverse resonant frequency method (ASTM C 215). The results of these measurements are shown in Figure 4. Also shown in Figure 4 are theoretical lines for several values of Poisson's ratio as computed from Eq. 2. The line that best fits the plotted points coincides with the theoretical line for Poisson's ratio equal to 0.26. Fairly reasonable estimates of the dynamic moduli could have been made from the values of $V_c^2\rho$ if 0.26 had been assumed for the Poisson's ratio of all beams. The standard deviation and the coefficient of variation for the prediction errors, assuming 0.26 for Poisson's ratio, are given in Table 1.

Variations in measured values obtained on replicate beams from the same concrete batch are due to both measurement errors and beam variations, and these variations might be considered a limiting value for any predicting technique. Thus, an analysis of variance was made for the values of dynamic modulus and the values of $V_c^2\rho$ for the 36 beams cast from the 12 different batches to determine the standard deviations and the coefficients of variation for these 2 parameters within a concrete batch. These values are also given in Table 1.

The prediction errors are not excessively large. Thus, there is substantial agreement between the resonant frequency method of determining dynamic elastic modulus (ASTM C 215) and values derived from velocity and unit weight measurements. From the following equation (Eq. 2 with Poisson's ratio equal to 0.26), a compressional wave velocity observation, in combination with a corresponding value of unit weight, can be utilized to estimate the dynamic elastic modulus as would be measured by using the transverse resonant frequency method:

$$\hat{E}_r = V_c^2 W/5,670 \tag{3}$$

Figure 4. Relation of dynamic modulus E_r measured by transverse resonant frequency method and quantity $V_c^2 \rho$ determined from laboratory measurements.

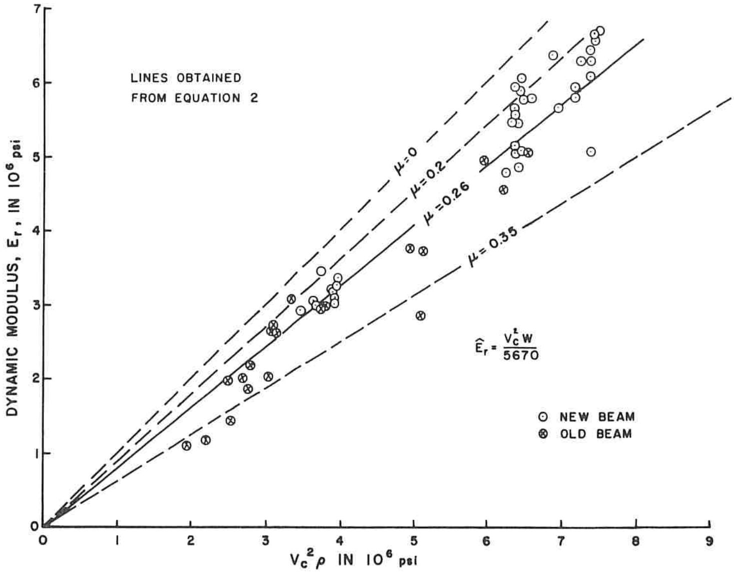


Figure 5. Relation of chord modulus E_c measured by stress-strain observations and quantity $V_c^2 \rho$ determined from laboratory measurements.

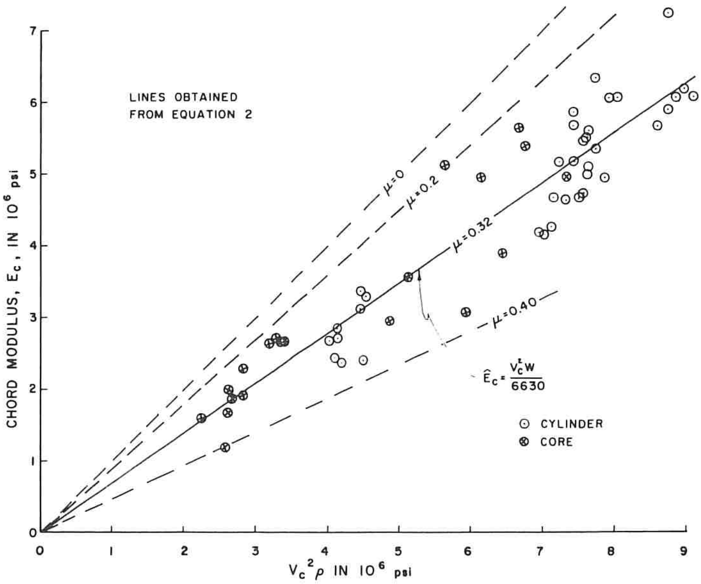


Table 1. Errors associated with moduli and quantity $V_c^2 \rho$.

Modulus	Error	Number of Tests	Standard Deviation (10^6 psi)	Coefficient of Variation (percent)
Dynamic	Prediction error (E_r - 0.817 $V_c^2 \rho$)	57	0.411	9.7
	Within batch replication error in E_r	36	0.334	6.5
	Within batch replication error in $V_c^2 \rho$	36	0.108	1.8
Chord	Prediction error (E_c - 0.699 $V_c^2 \rho$)	56	0.503	12.0
	Within batch replication error in E_c	36	0.348	7.3
	Within batch replication error in $V_c^2 \rho$	36	0.164	2.4

where

$\hat{E}_r$ = estimated dynamic modulus, in psi;
V_c = compressional wave velocity, in fps; and
W = unit weight, in lb/ft^3

Because the dynamic modulus is generally thought to be indicative of deterioration due to freeze-thaw cycles (ASTM C 290 and C 291), it follows that dynamic modulus values estimated from observations of the compressional wave velocity should be similarly indicative of deterioration within a bridge deck.

Relation of Velocity to Chord Modulus

The theoretically based relationship of $V_c^2\rho$ to elastic (chord) modulus was also examined from measurements made on 56 cylindrical specimens. Thirty-six of the specimens were cylinders 6 in. in diameter and 12 in. high. Three replicate specimens were cast from each of the previously mentioned 12 batches of concrete. The remaining 20 specimens were cores of various origins. Velocities were determined by the timing-through technique, unit weights were determined from the weights and dimensions of the specimens, and the chord moduli, E_c, were determined by stress-strain observations (ASTM C 469).

Figure 5 shows a plot of the chord modulus versus $V_c^2\rho$. The line that best fits the plotted points coincides with the theoretical line for Poisson's ratio equal to 0.32. Fairly reasonable estimates of the chord moduli could have been made from values of $V_c^2\rho$ if 0.32 had been assumed for the Poisson's ratio of all specimens. The apparent increase in Poisson's ratio over the value of 0.26 previously found is attributed to the fact that the chord modulus is usually found to be substantially smaller than the dynamic modulus. The standard deviation and the coefficient of variation for the prediction errors, assuming 0.32 for Poission's ratio, are given in Table 1. Also given are similar values for the within-batch replication errors for both the chord modulus and $V_c^2\rho$.

Again there is substantial agreement between the 2 methods of determining moduli. Velocity and unit weight can be used in the following equation (Eq. 2 with Poisson's ratio equal to 0.32) to estimate the chord modulus as would be determined with stress-strain measurements (ASTM C 469):

$$\hat{E}_c = V_c^2W/6{,}630 \qquad (4)$$

where

$\hat{E}_c$ = estimated chord modulus, in psi;
V_c = compressional wave velocity, in fps; and
W = unit weight, in lb/ft^3.

Thus, observations of compressional wave velocities should make it possible to rank the concrete at a number of locations with respect to the chord modulus. To the degree that a decrease in chord modulus indicates the occurrence of quality deterioration, the velocity measurement can serve to locate the extent of this deterioration.

Relation of Velocity to Compressive Strength

The relation between the compressional wave velocity and compressive strength was explored by measuring the ultimate compressive strength (ASTM C 39) of the 56 air-dried cylindrical specimens used for the chord modulus determinations.

No consistent relationship was found among all the cylindrical specimens, but separate tendencies were noted for the velocity to increase with strength within each group of cast cylinders containing a given type of coarse aggregate. Cores taken from beams were consistently higher in strength than cylinders cast from the same concrete batch, although their modulus values and their velocities were substantially alike.

Although no usable relation could be established for estimating the strength of all the cylindrical specimens from the measured velocities, the consistent trends for velocity to increase with modulus for all concretes tested and to increase with strength within groups having similar composition indicate that velocity measurements, utilized with discretion, are generally indicative of the quality of the concrete.

PORTABLE VELOCITY-MEASURING INSTRUMENT

A velocity-measuring system particularly adapted for use in the field was developed by utilizing the experience gained with the laboratory instrumentation described earlier. The basic considerations for its design were portability, accuracy, freedom from coupling errors, and convenience of operation. Because the instrument is intended for use on bridge decks, pavement slabs, and other concrete structures that may have only one flat surface accessible, it is based on the timing-along principle. The effect of time delays within the transducers themselves, or in the coupling of the transducers to the concrete, is minimized in this system by using an array of 2 transmitters and 2 receivers as shown in Figure 6. This array permits waves to be propagated from left to right by using the left transmitter or from right to left by using the right transmitter. Time of travel between the 2 receiving transducers, which are spaced 1 ft apart, is observed first for one direction of travel and then for the opposite direction. The 2 observed time intervals are then averaged to obtain a value that is substantially independent of any time delay in the coupling of either receiving transducer. Any excess delay in one of these couplings will lengthen the observed time interval for waves traveling in one direction but will diminish the observed interval by a like amount for oppositely traveling waves.

Timing is accomplished by separately observing the first zero crossing of each received signal on one trace of a dual-trace oscilloscope and setting an appropriately shaped voltage step to occur at the corresponding instant on the second trace. The appearance of this oscilloscope display is shown in Figure 7. When this matching has been done for both received signals, the time interval between the 2 voltage steps has been set equal to the time interval between the wave arrival at the 2 receivers. The 2 voltage steps are utilized respectively to start and stop a time-interval counter having a digital display readable to the nearest 0.1 μsec. A single switch on the control panel determines the direction of wave travel and selects which of the received signals is displayed on the oscilloscope.

The complete instrument comprises the probe, the control unit, and the oscilloscope as shown in Figure 8. The apparatus is intended primarily for operation from the tailgate of a station wagon and is powered, through an inverter, from the vehicle battery.

The probe, which is weighted to 30 lb to provide good coupling, is attached to the control unit through a flexible cable. Electrical coupling between the transmitting and receiving transducers is minimized by employing magnetostrictive transmitters with piezoelectric receivers. Individual pulse generators are mounted directly on the probe above each of the transmitting transducers, and receiving preamplifiers are mounted adjacent to the receivers. Acoustic coupling through the frame of the probe is made slow compared with the travel time in concrete by constructing the frame of low-velocity plastic material.

Coupling of the transducers to the concrete surface, particularly for somewhat rough or uneven surfaces, is facilitated by suspending the transmitting transducers from the frame in a flexible manner and by providing telescopic mountings for the receivers. Also, the receiving transducers themselves are permitted to swivel in ball joints at the bottom of their telescopic housings and, thus, adjust themselves to the local surface irregularities. A grease or starch couplant is placed on the contact surfaces of the probe.

In operation it has been found that a second operator can usually reproduce the observed time intervals to within about 0.4 μsec with the probe remaining stationary. Much larger variations are generally encountered when the probe is moved a few inches because of the inherent nonuniformity of the material. Variability of the time delay in the couplings of the 2 receivers has been found to range from 0 to as much as 4 μsec at some locations, but, as mentioned, averaging the observations for 2 directions of travel cancels this effect. Accordingly, the accuracy of this velocity meter appears to be limited principally by its readability and by the variability within the concrete. For an observed travel time of 60 μsec/ft, which corresponds to a velocity of 16,700 fps, the readability of 0.4 μsec represents about a 0.7 percent error; and for a speed of 100 μsec/ft, which corresponds to 10,000 fps, it is a 0.4 percent error. The inaccuracy might be as much as 10 times larger if the coupling delays were not averaged out.

Figure 6. Four tranducers for velocity measurement using timing-along technique in an array that propagates waves alternately in 2 directions and minimizes coupling delay errors.

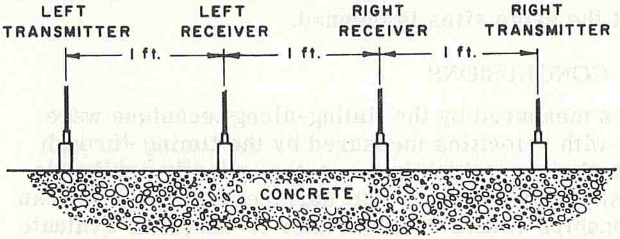


Figure 7. Representations of typical oscilloscope display showing received wave with 3 successive adjustments of voltage step used for timing wave arrival.

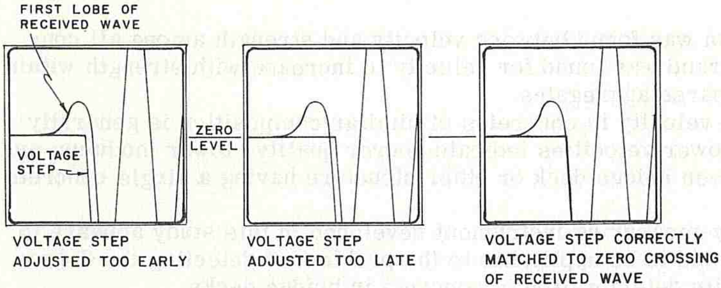
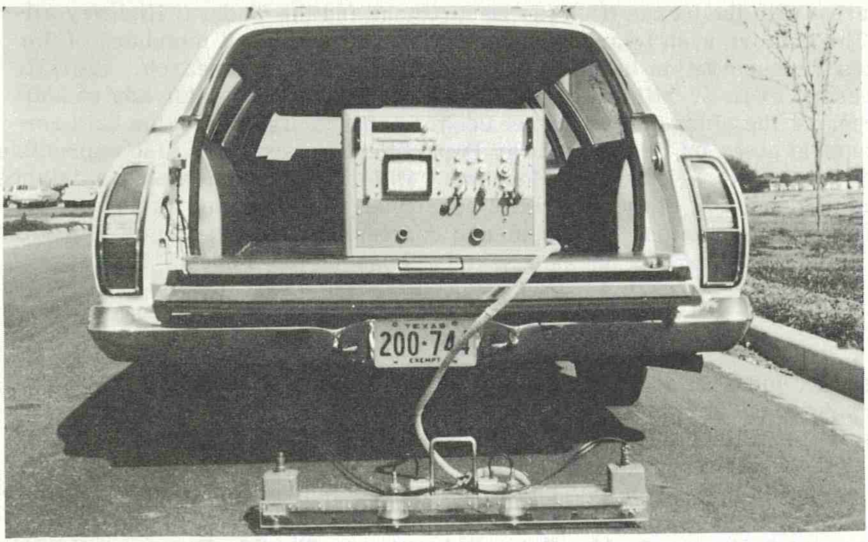


Figure 8. Field operation of portable velocity measuring instrument.



A limited number of measurements have been made with this instrument in the field on bridge decks. In general the system has proved fieldworthy, and its measurements appear to have adequate accuracy and resolution. It appears to be stable, rugged, and simple to operate. The velocities observed on the few bridges tested were all representative of good quality concrete. Further field evaluation of this instrument together with supplementary strength tests at the same sites is planned.

CONCLUSIONS

1. Compressional wave velocities measured by the timing-along technique were found to be in substantial agreement with velocities measured by the timing-through technique. Thus, the timing-along technique, which is more conveniently applicable on bridge decks and other concrete structures where 1 flat surface is accessible, can be utilized in connection with relationships established by either technique to evaluate the velocity-related properties of concrete in situ.
2. The compressional wave velocity and unit weight can be used to estimate the dynamic modulus of concrete as it would be determined by the transverse resonant frequency method (ASTM C 215).
3. The compressional wave velocity and unit weight can be used to estimate the chord modulus of concrete as determined from stress-strain measurements (ASTM C 469).
4. No consistent relation was found between velocity and strength among all concretes tested; however, a trend was found for velocity to increase with strength within concretes containing like coarse aggregates.
5. Compressional wave velocity in concretes of similar composition is generally related to their quality. Slower velocities indicate poorer quality, lower modulus, and diminished strength on a given bridge deck or other structure having a single concrete batch design.
6. The portable velocity-measuring instrument developed in this study appears to fulfill its design objectives and to be applicable to the problem of detecting the degree and the areal extent of quality deterioration of concrete in bridge decks.

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NONCONTACT MEASUREMENTS OF FOUNDATIONS AND PAVEMENTS WITH SWEPT-FREQUENCY RADAR

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Radar waves will be partially reflected whenever they encounter a change in electrical properties of the material in which they are traveling. Because highway profiles are formed of layered materials whose electrical properties differ, reflectance curves, as measured with a swept-frequency radar system, will contain information relating to the physical properties and the thicknesses of these layers. Analytical methods were developed to extract values for the electrical properties and layer thicknesses from the reflectance curves. Those methods are demonstrated on theoretical reflectance curves from typical profiles of an asphalt highway and a concrete highway. Swept-frequency radar measurements from either an air platform or a land vehicle show much promise for reducing both costs and time in the testing of old construction foundations and in the control of new construction.

• FOR A number of years the U. S. Army Engineer Waterways Experiment Station has conducted studies in the microwave spectral region to determine the feasibility of utilizing microwave techniques as a means of rapidly acquiring environmental data remotely. In these laboratory studies, the environmental parameters considered were limited for the most part to those dealing with soils, i.e., soil moisture content, soil composition, and layering profiles. The results of these laboratory studies indicated the following:

1. Imaging radar systems are not generally suitable for directly obtaining quantitative information on soil parameters. This type of radar system is sensitive only to the magnitude of the return signal and is not designed to discern individual components of the signal resulting from surface and subsurface reflections.
2. A number of different types of radar systems including frequency modulated, monopulse, and swept frequency can be used to distinguish between surface and subsurface returns. The swept-frequency radar system was chosen for further studies because it is the easiest to maintain and can be calibrated readily.
3. There is a direct correlation between a soil's dielectric constant and soil-water content.
4. Soil type per se has little effect on the return signal measured.
5. Swept-frequency radar techniques can be used to measure the optical thickness of layered materials and thus provide a means of estimating the physical thickness and the dielectric constant of that layer.

Based on the results of these laboratory studies and the layered structure of foundations and pavements, swept-frequency measurement techniques may provide an important remote-sensing tool in various construction procedures.

In this paper, the swept-frequency measurement technique and its mathematical development are described as they apply to foundations and pavements. In addition, the results of reflectance calculations on typical highway profiles are presented along with a summary of future applications.

REFLECTANCE ANALYSIS

Swept-Frequency Radar System Operation

A variable-frequency radar system can be used for layer measurements with relatively simple radar equipment consisting of a wide-band radar receiver and a transmitter with a variable-frequency output. This system can operate with either a pulsed or a continuous wave transmitter output. The speed of the frequency variation is not critical but must be slow enough so that the frequencies of the returns from the surface and the subsurface layers are essentially the same. In operation, the transmitter and receiver are positioned at vertical incidence to the surface to be measured and the frequency is varied. If a subsurface reflection occurs, the return received will decrease with frequency to a minimum, increase to a maximum, and then repeat the cycle over and over. The only measurement required is the frequency difference between either 2 adjacent maximums or 2 adjacent minimums. A block diagram of the variable-frequency radar system is shown in Figure 1, and the derivation of the depth-determination formula is given in the following. For the geometry shown in Figure 2, the return to the receiver will be a minimum. The phase shift between the surface and subsurface reflections is given by

$$\phi = (2X/\lambda)(2\pi) = 4\pi X/\lambda \tag{1}$$

and

$$\lambda = (c/\sqrt{\epsilon_r})(1/f) = c/\sqrt{\epsilon_r} f \tag{2}$$

where

ϕ = phase difference, rad;
X = depth of the medium, m;
λ = wavelength of wave in the medium, m;
c = speed of light, 300×10^6 m/sec;
ϵ_r = relative dielectric constant; and
f = frequency of radar wave, H_2.

The term $c/\sqrt{\epsilon_r}$ gives the speed of an electromagnetic wave in a medium with the conductivity term neglected (the conductivity term has little effect on wave velocity at frequencies above 200 MHz for soil samples thus far tested). From Eqs. 1 and 2,

$$\phi = [(4\pi X) f \sqrt{\epsilon_r}]/c \tag{3}$$

For a minimum, ϕ must be some odd multiple of π such as π, 3π, or 5π. This can be shown by $\phi = 2\pi n - \pi$, where n is an integer. There will be 2 adjacent minimums as the frequency is increased from f_1 to f_2 when n is increased by 1 unit.

$$\phi_1 = 2\pi n - \pi = [(4\pi X)(\sqrt{\epsilon_r})(f_1)]/c \tag{4}$$

$$\phi_2 = 2\pi(n + 1) - \pi = [(4\pi X)(\sqrt{\epsilon_r})(f_1)]/c \tag{5}$$

Subtracting Eq. 4 from Eq. 5 yields

$$\phi_2 - \phi_1 = 2\pi = [(4\pi X)(\sqrt{\epsilon_r})(f_2 - f_1)]/c \tag{6}$$

or

$$X = (1/2\sqrt{\epsilon_r})[(300 \times 10^6)/(f_2 - f_1)] \tag{7}$$

This derivation is good for either a pair of minimums at f_1 and f_2 or a pair of maximums at f_1 and f_2 because only the frequency difference, Δf, is used in the calculation for depth. From the formula, the only other variable besides the frequencies is that of relative dielectric constant, ϵ_r. The relative dielectric constant can be estimated from the average power reflectance of this variable-frequency radar system as in the following equation:

$$\epsilon_{r1} = \epsilon_{r2} \left[(1 + \sqrt{R})/(1 - \sqrt{R})\right]^2 \tag{8}$$

where

ϵ_{r1} = relative dielectric constant for material beneath the reflecting interface;
ϵ_{r2} = relative dielectric constant for material in which the incident wave is traveling (if material is air, $\epsilon_{r2} = 1$); and
R = power reflectance.

Calculation of Reflectance Coefficients

An insight to the problem of swept-frequency measurements can be gained by calculating the reflectance coefficients as a function of frequency for various layered profiles. The equations used for these calculations are extracted from transmission line theory with the following assumptions:

1. The electrical properties, relative dielectric constant ϵ_r, and conductivity σ are not frequency dependent over the range of interest—a valid assumption based on past work (1);
2. Transition zones between layers do not exist or are small enough to be neglected (thus a reflection is obtained at the interface between layers, e.g., air-soil interface, air-pavement interface, pavement-soil interface, or an interface between soil layers with differing electrical properties); and
3. Layers have homogeneous electrical properties.

One set of equations that can be used to calculate the reflectance of the layered profiles is as follows:

$$Z_{Ln} = Z_{on} \left\{\left[Z_{Ln+1} \text{COSH}(\gamma_n \ell_n) + Z_{on}/\text{SINH}(\gamma_n \ell_n)\right] / \left[Z_{on} \text{COSH}(\gamma_n \ell_n) + Z_{Ln+1} \text{SINH}(\gamma_n \ell_n)\right]\right\} \tag{9}$$

$$Z_{on} = \sqrt{\mu_n/\epsilon_n^*} \tag{10}$$

$$R = \left[(Z_{L1} - Z_{AIR})/(Z_{L1} + Z_{AIR})\right]^2 \tag{11}$$

where

Z_{Ln} = load impedance for nth layer;
Z_{on} = characteristic impedance for nth layer;
R = power reflectance;
γ = propagation factor;
ℓ = layer thickness;
μ = magnetic permeability;
ϵ^* = complex dielectric constant; and
Z_{AIR} = characteristic impedance for air (377 ohms).

Each layer has a characteristic impedance as calculated from Eq. 8. The characteristic impedance at the last layer of assumed infinite depth (the foundation) acts as the load impedance for the preceding layer. Equation 9 can then be used to transform the impedance through the layer to the bottom of the next succeeding layer. The transformed impedance, in turn, acts as a load impedance for the next layer and so on until

Figure 1. Variable-frequency radar system.

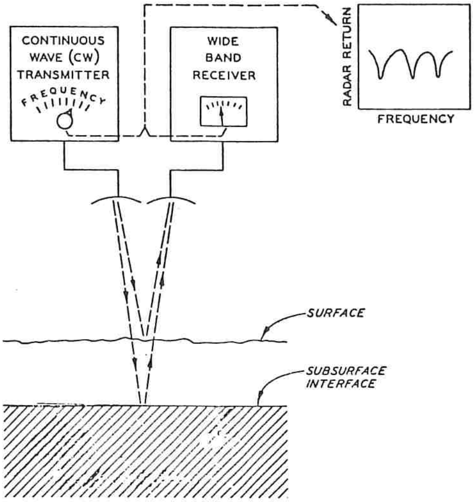


Figure 2. Wave phase change in soil.

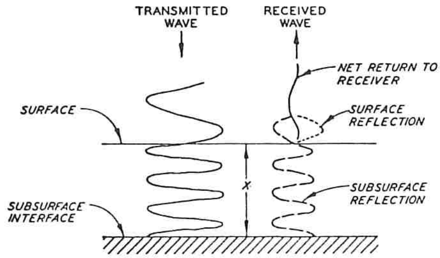
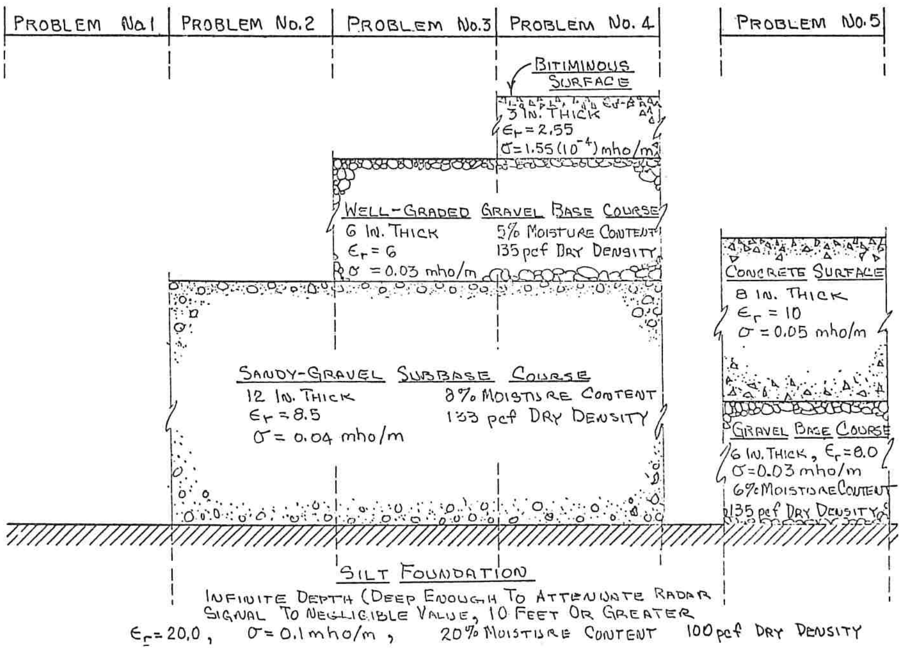


Figure 3. Selected highway profiles.



the surface is reached. The air-surface mismatch will govern the power reflected back to the radar receiver. The reflectance coefficient for this situation is calculated from Eq. 11.

Because of the repetitive nature of the calculation, a considerable amount of time and effort can be saved by using a computer. Programs were written for a GE 400 series computer, and the results of calculations for some typical foundations and pavements are given in the next section. The impedance equations are transcendental, and thus a direct solution for the input parameters in terms of the calculated reflectance values is not possible. Simplifying assumptions must be made in the analysis of the reflectance curves such that the problem is broken into a number of smaller subproblems. These subproblems can then be solved and the results combined for a final solution.

RESULTS OF CALCULATION

A number of highway profiles were selected for further study. These profiles were a series of layers in which each new profile was formed by adding another layer on top of the previous profile. This would be similar to that expected during the actual construction of a highway if one were to take a measurement after each new construction phase. The profiles for the problems are shown in Figure 3 where the electrical properties were estimated from the physical properties of each layer. Calculated values for power reflectance were obtained from the procedures discussed earlier and are shown versus frequency in Figures 4 and 5 for each of the problems shown in Figure 3. These power reflectance values should be the same as those obtained from actual swept-frequency radar measurements after corrections are made to allow for surface irregularities, minor transition zones, and small nonhomogeneous zones within the layers. The results of the following data analysis are given in Table 1.

Problem 1

The profile for this problem was the silt highway foundation by itself. Its reflectance curve starts at a value of 0.4073 at 500 MHz and decreases to 0.4028 at 2,400 MHz (Fig. 4). The following conclusions can be drawn from this curve:

1. The uniformity of the curve (no cyclic patterns) denotes a deep homogeneous material with no layered structure.
2. The reflectance values are less influenced by the conductivity value at high frequencies than at low frequencies. The curve appears to be asymptotic to a value of 0.4027. This value can be used to estimate the relative dielectric constant as shown in the following, and this in turn can be used to estimate the volumetric water content of the soil:

$$\epsilon_r = [(1 + \sqrt{R})/(1 - \sqrt{R})]^2 = [(1 + \sqrt{0.4027})/(1 - \sqrt{0.4027})]^2 = 20.0$$

The conductivity can be calculated by using the reflectance curve data at the low frequencies and the asymptotic reflectance value. Any differences are due to the conductivity term.

Problem 2

The profile for this problem was the silt foundation covered by a 12-in. subbase course of sandy gravel. Its reflectance curve is characterized by a cyclic pattern throughout the 500- to 2,400-MHz frequency range and of a lower amplitude level than for problem 1 (Fig. 4). The average reflectance decreases with increasing frequency in a manner similar to that of problem 1. The following conclusions can be drawn from the curve:

1. The magnitude of the oscillations can be used to estimate the electrical properties of the subsurface material.
2. The magnitude of the surface reflectance can be used to estimate the relative dielectric constant of the surface soil as in problem 1. The power reflectance curve

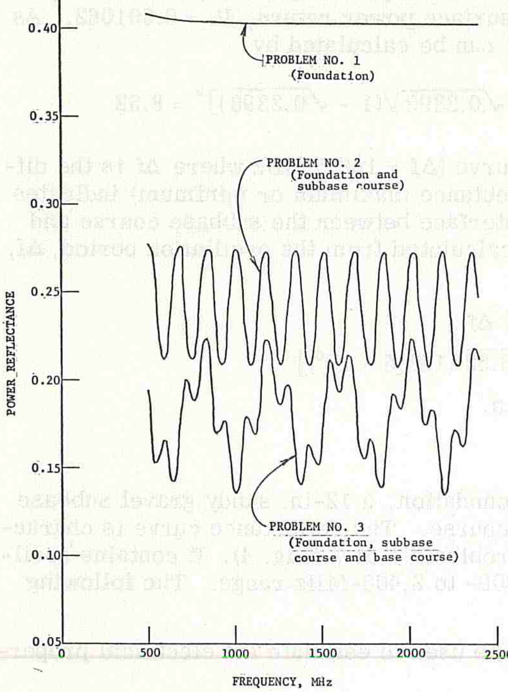


Figure 4. Reflectance curves for base materials.

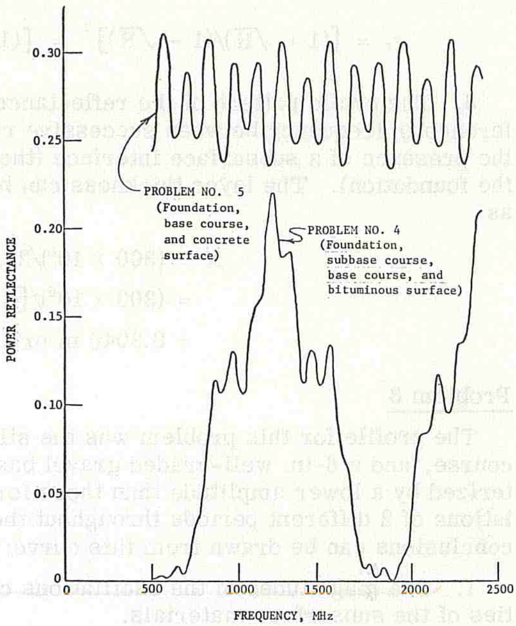


Figure 5. Reflectance curves for pavements.

Table 1. Summary of analysis results.

Type of Highway	Layer	Original Input Data E_r	Original Input Data Layer Thickness (in.)	Problem	Analysis Result E_r	Analysis Result Layer Thickness (in.)
Bituminous	Surface	2.55	3.0	4	2.60	3.05
	Base	6.0	6.0	4		6.01
				3	6.03	6.02
	Subbase	8.5	12.0	4		12.30
				3		12.01
				2	8.52	12.00
	Foundation	20.0		1	20.0	
Concrete	Surface	10	8.0	5	9.97	8.02
	Base		6.0	5		6.05

can (on the basis of voltage interactions) be separated into a constant surface power reflection and an oscillating subsurface reflection component as follows: surface power reflectance, R = 0.2398, and normalized subsurface power return, P_B = 0.001062. As in problem 1, the relative dielectric constant can be calculated by

$$\epsilon_r = [(1 + \sqrt{R})/(1 - \sqrt{R})]^2 = [(1 + \sqrt{0.2398})/(1 - \sqrt{0.2398})]^2 = 8.52$$

3. The cyclic pattern of the reflectance curve (Δf = 168.5 MHz where Δf is the difference in frequency between successive reflectance maximum or minimum) indicates the presence of a subsurface interface (the interface between the subbase course and the foundation). The layer thickness can be calculated from the oscillation period, Δf, as

$$\begin{aligned} X &= (300 \times 10^6)/2\sqrt{\epsilon_r}\ \Delta f \\ &= (300 \times 10^6)/[2\sqrt{8.52}\ (168.5 \times 10^6)] \\ &= 0.3046 \text{ m or } 12 \text{ in.} \end{aligned}$$

Problem 3

The profile for this problem was the silt foundation, a 12-in. sandy gravel subbase course, and a 6-in. well-graded gravel base course. The reflectance curve is characterized by a lower amplitude than those for problems 1 or 2 (Fig. 4). It contains oscillations of 2 different periods throughout the 800- to 2,400-MHz range. The following conclusions can be drawn from this curve:

1. The magnitudes of the oscillations can be used to estimate the electrical properties of the subsurface materials.

2. The magnitude of the surface reflectance can be used to estimate the relative dielectric constant of the surface soil as in problems 1 and 2. The smaller amplitude of this curve compared to those of problems 1 and 2 denotes a smaller relative dielectric constant. Separating the total reflectance curve into 2 components gives surface power reflectance, R = 0.1775, and normalized subsurface power return, P_B = 0.002545. The surface relative dielectric constant can then be calculated as

$$\epsilon_r = [(1 + \sqrt{R})/(1 - \sqrt{R})]^2 = [(1 + \sqrt{0.1775})/(1 - \sqrt{0.1775})]^2 = 6.03$$

3. The cyclic patterns of 2 different periods (Δf_1 = 118.6 MHz and Δf_2 = 400 MHz) indicate the presence of 2 subsurface interfaces. The short period oscillation (Δf_1 = 118.6 MHz) is associated with the interface between the subbase course and the foundation. The long period oscillation (Δf_2 = 400 MHz) is associated with the interface between the base course and the subbase course. The layer thicknesses, X_1 and X_2, can be calculated as follows by using the relative dielectric constants of 6.03 and 8.52 for the first (top) and second layers respectively as computed previously. For the top layer (base course) thickness

$$\begin{aligned} X_1 &= (300 \times 10^6)/2\sqrt{\epsilon_r}\ \Delta f \\ &= (300 \times 10^6)/[2\sqrt{6.03}\ (400 \times 10^6)] \\ &= 0.53 \text{ m or } 6.02 \text{ in.} \end{aligned}$$

For the second layer (subbase course) thickness, the optical depth to the second interface must first be calculated.

$$\begin{aligned} X_0 &= (300 \times 10^6)/2\Delta f \\ &= (300 \times 10^6)/[2(118.6 \times 10^6)] \\ &= 1.265 \text{ m} \end{aligned}$$

The second layer thickness, X_2, is then computed by

$$X_2 = (X_0 - X_1 \sqrt{\epsilon_{r1}})/\sqrt{\epsilon_{r2}}$$
$$= (1.265 - 0.153\sqrt{6.03})/\sqrt{8.52}$$
$$= 0.3048 \text{ m or } 12.01 \text{ in.}$$

Problem 4

The profile for this problem was the silt foundation, a 12-in. sandy-gravel subbase course, a 6-in. well-graded gravel base course, and a 3-in. bituminous surface. This reflectance curve is characterized by a still lower amplitude than those for problems 1, 2, or 3 (Fig. 5). It has oscillations of 3 different periods throughout the 500- to 2,400-MHz range. The following conclusions can be drawn from this curve:

1. The magnitude of the oscillation can be used to estimate the electrical properties of the subsurface materials.

2. The magnitude of the surface reflectance can be used to estimate the relative dielectric constant of the surface soil as before. The reflectance curve can be separated into 2 components (in this case, equal components): surface power reflectance, $R = 0.055$, and normalized subsurface power return, $P_B = 0.055$. The surface relative dielectric constant can be calculated as

$$\epsilon_r = [(1 + \sqrt{R})/(1 - \sqrt{R})]^2 = [(1 + \sqrt{0.055})/(1 - \sqrt{0.055})]^2 = 2.6$$

3. The cyclic patterns of 3 different periods ($\Delta f_1 = 106.2$ MHz, $\Delta f_2 = 300$ MHz, and $\Delta f_3 = 1{,}200$ MHz) indicate the presence of 3 subsurface interfaces. The short period oscillation ($\Delta f_1 = 106.2$ MHz) is associated with the interface between the subbase course and the foundation, the medium period oscillation ($\Delta f_2 = 300$ MHz) is associated with the interface between the base course and the subbase course, and the long period oscillation is associated with the interface between the bituminous material and the base course. The layer thicknesses, X_1, X_2, and X_3, can be calculated as follows by using the relative dielectric constant values previously calculated: 2.6, 6.03, and 8.52 for the top, middle, and bottom layers respectively. The top layer (bituminous surface) thickness, X_1, is

$$X_1 = (300 \times 10^6)/2\sqrt{\epsilon_r}\ \Delta f$$
$$= (300 \times 10^6)/[2\sqrt{2.6}\,(1{,}200 \times 10^6)]$$
$$= 0.0775 \text{ m or } 3.05 \text{ in.}$$

The optical depth to the second interface is

$$X_{02} = (300 \times 10^6)/2\Delta f$$
$$= (300 \times 10^6)/[2(300 \times 10^6)]$$
$$= 0.5 \text{ m}$$

The second layer (base course) thickness X_2 is

$$X_2 = (X_{02} - X_1 \sqrt{\epsilon_{r1}})/\sqrt{\epsilon_{r2}}$$
$$= (0.5 - 0.0775\sqrt{2.6})/\sqrt{6.03}$$
$$= 0.1527 \text{ m or } 6.01 \text{ in.}$$

The optical depth to the third interface is

$$X_{03} = (300 \times 10^6)/2\Delta f$$

$$= (300 \times 10^6)/[2(106.2 \times 10^6)]$$
$$= 1.412 \text{ m}$$

The third layer (subbase course) thickness X_3 is

$$X_3 = (X_{03} - X_{02})/\sqrt{\epsilon_{r3}}$$
$$= (1.412 - 0.5)/\sqrt{8.52}$$
$$= 0.3123 \text{ m or } 12.3 \text{ in.}$$

Problem 5

The profile for this problem was slightly different from those of problems 2, 3, and 4. The foundation is silt as before but the subbase course has been eliminated. A 6-in. base course of gravel covers the foundation that in turn is capped by an 8-in. layer of concrete. The reflectance curve from this profile is characterized by a general amplitude between those of problems 1 and 2 (Fig. 5). It has a somewhat random cyclic appearance that can be resolved into oscillations with 2 different periods of nearly constant amplitude over the 500- to 2,400-MHz range. The following conclusion can be drawn from this curve:

1. The magnitude of the oscillations can be used to estimate the electrical properties of the subsurface materials.
2. The magnitude of the surface reflectance can be used to estimate the relative dielectric constant of the concrete surface. Separating the reflectance curve into 2 components gives surface power reflectance, $R = 0.269$, and normalized subsurface power return, $P_B = 0.001303$. The surface (concrete) relative dielectric constant can be calculated as

$$\epsilon_r = [(1 + \sqrt{R})/(1 - \sqrt{R})]^2 = [(1 + \sqrt{0.269})/(1 - \sqrt{0.269})]^2 = 9.97$$

3. The cyclic patterns of 2 different periods ($\Delta f_1 = 139.13$ MHz and $\Delta f_2 = 233.33$ MHz) indicate the presence of 2 subsurface interfaces: between the base course and the foundation (Δf_1) and between the concrete and the base course (Δf_2). The layer thicknesses X_1 and X_2 for the concrete and base course can be calculated by using the relative dielectric constant value of 9.97 for concrete previously calculated and selecting a relative dielectric constant of 8.0 for the base course. The concrete layer thickness X_1 is

$$X_1 = (300 \times 10^6)/2\sqrt{\epsilon_{r1}}\,\Delta f$$
$$= (300 \times 10^6)/[2\sqrt{9.97}\,(233 \times 10^6)]$$
$$= 0.2036 \text{ m or } 8.02 \text{ in.}$$

The optical depth to the second interface is

$$X_0 = (300 \times 10^6)/2\Delta f$$
$$= (300 \times 10^6)/[2(139.13 \times 10^6)]$$
$$= 1.078 \text{ m}$$

The second layer (base course) thickness is

$$X_2 = (X_0 - X_1\sqrt{\epsilon_{r1}})/\sqrt{\epsilon_{r2}}$$
$$= (1.078 - 0.2036\sqrt{9.97})/\sqrt{8.0}$$
$$= 0.154 \text{ m or } 6.05 \text{ in.}$$

CONCLUSIONS

The following conclusions are based on the measurement procedure and the results of analysis reported in this paper:

1. The results of analysis demonstrate the feasibility of using swept-frequency techniques in estimating the properties and thicknesses of layered materials;
2. These results show promise for reducing time and money spent on testing old highways, landing strips, or other construction foundations (for example, swept-frequency techniques could be used to make measurements on layer thicknesses beneath the pavement wearing surface, to detect cavities or voids under the pavement surface, or to detect water pockets in the subsurface layers); and
3. The swept-frequency measurements may aid in the control of new construction (for example, these techniques could be used to rapidly evaluate the thickness of an asphalt surface layer placed on an old concrete highway, measure the uniformity of materials in both quality and thickness, or measure the depth to reinforcing steel in concrete).

REFERENCE

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FIELD INVESTIGATION OF CONCRETE QUALITY USING THE WINDSOR PROBE TEST SYSTEM

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Extensive application of the Windsor probe test system has been made in investigations of in-place compressive strengths of concrete and in determinations of concrete quality. The system drives with a constant energy a steel probe into concrete; the depth of probe penetration is a measure of resistance to penetration. This probe penetration is thus indicative of compressive strength. Comparisons of these compressive strengths with those of cores or cylinders have been made. The Windsor probe has been used in investigations of compressive strengths of reinforced concrete pipe, highway bridge piers, abutments, and pier caps whose strengths were in question; of concrete pavements whose strengths were suspected of being weakened by lightning strikes; and of concrete surfaces damaged by fire.

•AN INVESTIGATION of the Windsor probe test system as a means of determining the in-place compressive strength of concrete in a quick, relatively nondestructive manner was begun in 1967. The system consists of a driver and a steel probe. The probe is driven into the concrete by 600 ft-lb of energy imparted by an accurately loaded, center-fired cartridge. The probe tip is machined to pierce the aggregate as well as the cement-sand mixture, yielding an average strength determination. A locating template is also provided that allows positioning of the 3 triangular "shots" constituting a test, and the accompanying gage plates allow the averaging of the 3 probe heights above the concrete surface. Measurement of this averaged exposed height above the concrete surface is made by using a calibrated depth gage.

The actual compressive strength of the concrete is determined by using a table, provided by the manufacturer and shown in Figure 1, that relates exposed height of the probe to compressive strength of concrete. Several columns in the table cover the range of Mohs' hardness of the aggregate, and a Mohs' hardness kit is also supplied to allow this determination to be made in the field at the site of testing. A probe withdrawal kit is provided for use in removing test probes from the concrete. A hole remains with a diameter of about ¼ in. and an average depth of 1 in. in standard cured concrete. Although some spalling may occur around the hole, the system is relatively nondestructive when compared to a core drill.

FIELD TESTS AND RESULTS

An initial evaluation of the system evolved in conjunction with an investigation to determine whether reinforced, cement concrete pipe being produced met the material specifications of the Pennsylvania Department of Transportation. Relations to be investigated were (a) cylinder strength and concrete mixes, (b) cylinder strength and nonreinforced core strength, (c) cylinder strength and reinforced core strength, (d) cylinder strength and 3-edge bearing test, and (e) core strength and 3-edge bearing test.

For this investigation, a study of the application of the Windsor probe system was obvious. The investigation required that cores be taken from sections of the pipe under test (48-in. reinforced concrete pipe, class 5) and broken after 14 days. Corresponding strengths were determined on the cored pipe samples by using the Windsor system. Alterations to the system were made so that probes could be fired into the curved surface of the pipe. A hand-held fixture similar to a single, raised boss of the locating tem-

Figure 1. Values of compressive strength Mohs' hardness numbers for exposed Windsor probe height.

CONCRETE COMPRESSIVE STRENGTHS FOR USE IN WINDSOR PROBE TEST

EXPOSED PROBE (in.)	COMPRESSIVE STRENGTH (p.s.i.) MOH'S NO. 3	MOH'S NO. 4	MOH'S NO. 5	MOH'S NO. 6	MOH'S NO. 7
1.000	-	-	-	-	-
1.100	1000	-	-	-	-
1.125	1167	-	-	-	-
1.150	1333	-	-	-	-
1.175	1500	-	-	-	-
1.200	1667	900	-	-	-
1.225	1833	1075	-	-	-
1.250	2000	1250	-	-	-
1.275	2167	1425	-	-	-
1.300	2334	1600	-	-	-
1.325	2500	1775	900	-	-
1.350	2667	1950	1100	-	-
1.375	2834	2125	1300	-	-
1.400	3000	2300	1500	-	-
1.425	3167	2475	1687	-	-
1.450	3334	2650	1875	1000	-
1.475	3500	2725	2062	1200	-
1.500	3667	3000	2250	1400	-
1.525	3834	3175	2437	1600	-
1.550	4000	3350	2625	1800	814
1.575	4167	3525	2812	2000	1032
1.600	4334	3700	3000	2200	1250
1.625	4500	3875	3187	2400	1468
1.650	4667	4050	3375	2600	1687
1.675	4834	4225	3562	2800	1906
1.700	5000	4400	3750	3000	2125
1.725	5167	4575	3937	3200	2343
1.750	5334	4750	4125	3400	2562
1.775	5500	4925	4312	3600	2781
1.800	5667	5100	4500	3800	3000
1.825	5834	5275	4687	4000	3218

EXPOSED PROBE (in.)	COMPRESSINE STRENGTH (p.s.i.) MOH'S NO. 3	MOH'S NO. 4	MOH'S NO. 5	MOH'S NO. 6	MOH'S NO. 7
1.850	6000	5400	4875	4200	3437
1.875	6167	5625	5062	4400	3656
1.900	6334	5800	5250	4600	3875
1.925	6500	5975	5437	4800	4093
1.950	6667	6150	5625	5000	4312
1.975	6833	6325	5812	5200	4531
2.000	7000	6500	6000	5400	4750
2.025	7167	6675	6187	5600	4968
2.050	7334	6850	6375	5800	5187
2.075	7500	7025	6562	6000	5406
2.100	7667	7200	6750	6200	5625
2.125	7833	7375	6937	6400	5843
2.150	8000	7550	7125	6600	6062
2.175	8167	7725	7312	6800	6281
2.200	8334	7900	7500	7000	6500
2.225	8500	8075	7687	7200	6718
2.250	8667	8250	7875	7400	6937
2.275	8834	8425	8062	7600	7156
2.300	9000	8600	8250	7800	7375
2.325	9167	8775	8437	8000	7593
2.350	9334	8950	8625	8200	7812
2.375	9500	9125	8812	8400	8031
2.400	9667	9300	9000	8600	8250
2.425	9334	9475	9187	8800	8468
2.450	10000	9650	9375	9000	8687
2.475	-	9825	9562	9200	8906
2.500	-	10000	9750	9400	9125
2.525	-	-	9937	9600	9353
2.550	-	-	10125	9800	9562
2.575	-	-	-	10000	9781
2.600	-	-	-	-	10000

NOTES:

1. If in doubt as to Moh's Number, confirm with Windsor Scratch Test Set.
2. Moh's Hardness Rating: No. 3 - Limestone (Calcite)
   4 - Limestone (Fluorite)
   5 - Limestone (Apatite)
   6 - Trap Rock (Microcline)
   7 - Gravel (Quartz)
3. Control Strength: A.S.T.M. 39 Cylinders
   A.S.T.M. 42 Cores

Table 1. Average core and Windsor probe strengths in tests of pipe produced by various plants.

Plant	Core (psi)	Windsor (psi)	Plant	Core (psi)	Windsor (psi)
1	6,672	7,550	4	6,699	6,525
2	4,857	5,400	5	5,936	6,500
3	6,787	6,600	6	6,453	5,975

Table 2. Core and Windsor probe strengths in lightning-damage tests.

Core	Core (psi)	Windsor (psi)	Core	Core (psi)	Windsor (psi)
1	3,451	3,475	4	4,746	4,833
2	3,998	3,475	5	3,748	4,082
3	3,884	—			

plate was made, and tests consisted of individual shots at sites near the core locations instead of the 3-shot group used on flat test surfaces. Reliable exposure heights for the individual probe can be made by using the apparatus provided with the system.

Tests were run at 6 pipe manufacturing plants where various manufacturing methods were used. Because this application was made prior to the development of the table shown in Figure 1, a curve supplied by the manufacturer of exposed probe height versus compressive strength was used (Fig. 2). This curve was supposedly developed by the U.S. Bureau of Public Roads using several hundred controlled cylinder breaks. A comparison of the core strength and the Windsor probe strength is given in Table 1. Each core compression value given is an average of 12 core breaks, and each Windsor value is an average of approximately 25 shots with usually 2 shots taken near each core location. These data reduce to a mean core strength of 6,234 psi with a standard deviation of ±677 psi and a coefficient of variation of 10.9 percent. The Windsor results show a mean strength of 6,425 psi with a standard deviation of ±681 psi and a coefficient of variation of 10.6 percent. Tests with the probe were taken 1 to 2 weeks after the core tests; the cores were tested at 14 days age.

The favorable results of the initial application of the Windsor system to the concrete pipe tests opened possibilities of its use in other concrete testing areas.

When the question arose as to the strength of certain sections of concrete pavement on the Pennsylvania Turnpike at Harrisburg and on Interstate 83 across the Susquehanna River, the Windsor system was again applied to obtain a possible answer; but again it was used in conjunction with cored samples for comparison. For these tests, the Mohs' hardness data shown in Figure 1 were used to determine the Windsor strengths. The Turnpike tests yielded a Windsor probe value, averaged for 6 test sites, of 6,098 psi with a standard deviation of ±477 psi. Five cores taken in close proximity to the probe tests showed an average compressive strength of 5,255 psi with a standard deviation of ±567 psi. On the Interstate bridge, the average compressive strengths were 5,100 psi with a standard deviation of ±648 psi for 6 Windsor test sites and 5,515 psi with standard deviation of ±1,390 psi for only 2 cores.

When the results of these tests became known to Department of Transportation personnel, requests for the use of the Windsor probe system increased. Many of these applications required only that a "ball-park" figure of compressive strength be determined, for in some cases cores were difficult or impossible to obtain.

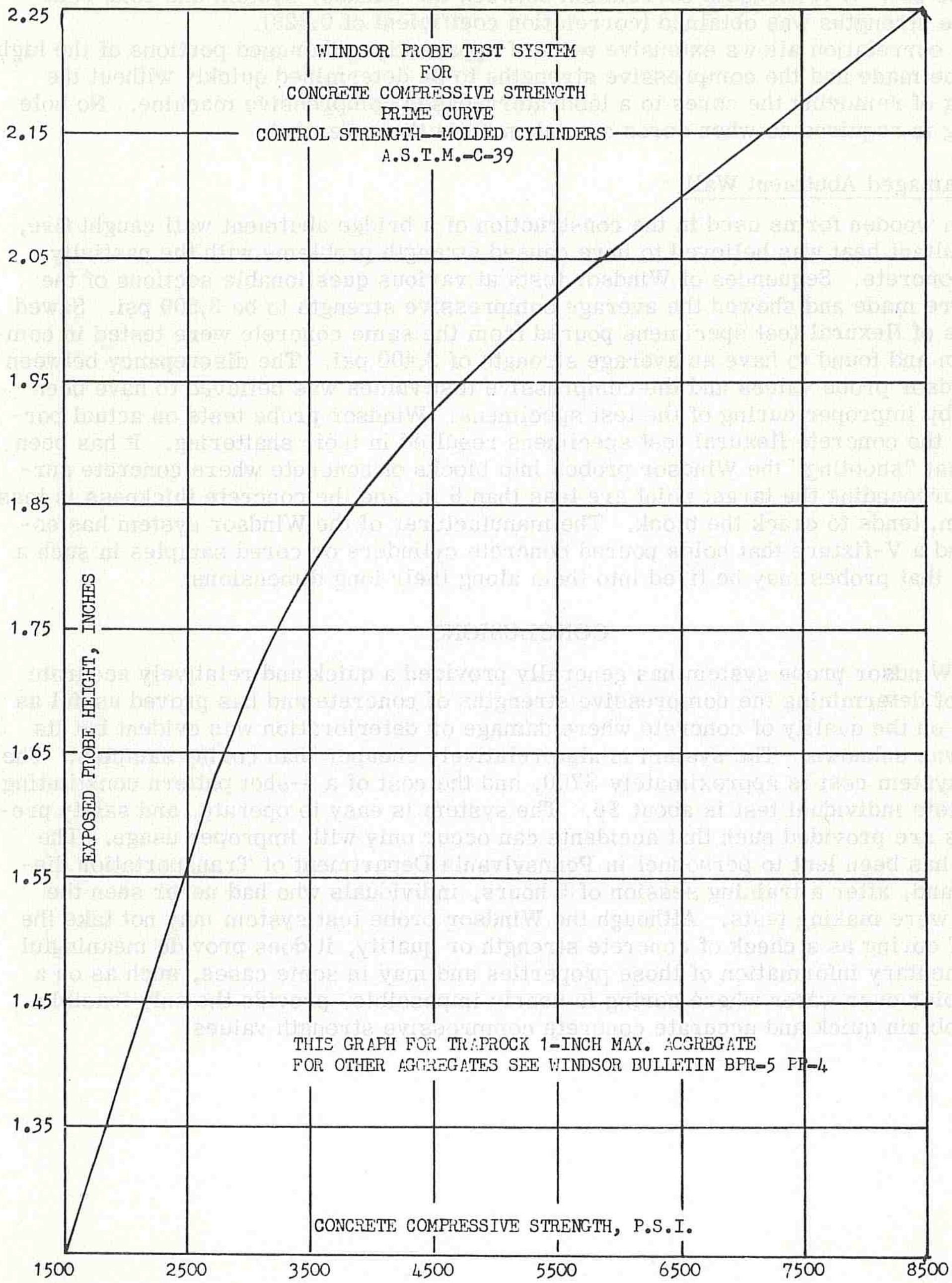
Pier Erosion

One such application was the testing of vertical bridge piers on a section of a Schuylkill Expressway Bridge across the Schuylkill River in Philadelphia. A major portion of some piers had been eroded by the river, and a question as to the strength of other apparently good piers arose. More than 100 piers were individually tested. Good piers showed strengths exceeding 8,500 psi in some cases, and many piers that appeared visually good had strengths of less than 1,800 psi. Some piers were even found to be hollow as the steel Windsor probe went into the pier column completely such that no exposure height was measurable. Thus, the Windsor probe system provided a quick and relatively accurate check of the compressive strengths, where measurable, and the general quality of the piers.

Lightning Damage

The damage caused by lightning striking a section of a reinforced concrete highway resulted in an investigation of damage not visible from the pavement surface. Nondestructive techniques, including the Windsor probe, were used in areas other than where sections of pavement as large as 2.5 by 3.2 by 0.4 ft deep were blown out at joint locations to obtain an overall damage pattern. Five cores were taken from the 500-ft damaged area of pavement, and core 4 was used as a standard inasmuch as it was taken from an area of no visible damage. The results of the Windsor and the core compressive strengths are given in Table 2. The average compressive strength of the concrete cores was 3,986 psi with ±478 psi standard deviation; the average Windsor strength was 3,966 psi with ±558 psi standard deviation. Core 3 was not included because no

Figure 2. Compressive strength versus exposed Windsor probe height.



probe test was taken at this core location. The rms deviation of probe versus cores was ±325 psi. A reasonable correlation between the Windsor system and core compressive strengths was obtained (correlation coefficient of 0.829).

Such correlation allows extensive tests of apparently undamaged portions of the highway to be made and the compressive strengths to be determined quickly without the time lag of removing the cores to a laboratory-based compressive machine. No hole patching is required as when cores are taken from the pavement.

Fire-Damaged Abutment Wall

When wooden forms used in the construction of a bridge abutment wall caught fire, the resultant heat was believed to have caused strength problems with the partially cured concrete. Sequences of Windsor tests at various questionable sections of the wall were made and showed the average compressive strength to be 3,500 psi. Sawed samples of flexural test specimens poured from the same concrete were tested in compression and found to have an average strength of 2,400 psi. The discrepancy between the Windsor probe values and the compressive test values was believed to have been caused by improper curing of the test specimens. Windsor probe tests on actual portions of the concrete flexural test specimens resulted in their shattering. It has been noted that "shooting" the Windsor probes into blocks of concrete where concrete surfaces surrounding the target point are less than 8 in. and the concrete thickness is less than 6 in. tends to crack the block. The manufacturer of the Windsor system has established a V-fixture that holds poured concrete cylinders or cored samples in such a manner that probes may be fired into them along their long dimensions.

CONCLUSIONS

The Windsor probe system has generally provided a quick and relatively accurate means of determining the compressive strengths of concrete and has proved useful as a check on the quality of concrete where damage or deterioration was evident but its extent was unknown. The system is also relatively cheaper than coring samples. The initial system cost is approximately $700, and the cost of a 3-shot pattern constituting a complete individual test is about $6. The system is easy to operate, and safety precautions are provided such that accidents can occur only with improper usage. The system has been lent to personnel in Pennsylvania Department of Transportation districts, and, after a training session of 5 hours, individuals who had never seen the system were making tests. Although the Windsor probe test system may not take the place of coring as a check of concrete strength or quality, it does provide meaningful supplementary information of those properties and may in some cases, such as on a bridge pier over water where coring is nearly impossible, provide the only feasible way to obtain quick and accurate concrete compressive strength values.

IMPACT AND PENETRATION TESTS OF PORTLAND CEMENT CONCRETE

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Tests using a rebound device (Schmidt hammer) and a penetration device (Windsor probe test system) were performed at 4 ages on mortar specimens and on concretes made with 3 coarse aggregates and with 2 maximum sizes of aggregate. Cylinders, 6 by 12 in., also were cast and tested at the same ages. Comparisons of the test methods were made of their ability to detect significant differences in batches of concrete, aggregates, sizes of aggregates, successive ages of concrete, and top and bottom of test slabs. Results showed that more significant differences among concrete specimens were detected by the cylinder tests than by either hammer or probe measurements. Based on this study and others, it appears that, although both the Swiss hammer and the Windsor probe test system show a correlation with compressive strength, neither is sufficiently precise to give accurate compressive strength values. Either instrument can be used to investigate relative quality of different areas of concrete and to survey areas of deteriorated concrete. The Swiss hammer, however, is cheaper than the Windsor probe test system, less destructive to the concrete surface, and capable of providing a much greater number of tests in a given area.

•IN MARCH and April 1968, a test program was conducted in the laboratory of the Bureau of Public Roads (now Federal Highway Administration), the prime purpose of which was to study the extent to which the Windsor probe test system and the Schmidt hammer could be used to determine the strength of concrete and to determine the relative usefulness of the 2 methods. A full report on this research is to be published soon (1).

In general, our conclusions agree with those of other researchers. Although both rebound and probe measurements show a correlation with compressive strength, neither provides a precise determination of strength. Either can be used to assess relative strengths in different concretes or different areas of the same concrete, to survey a concrete surface to find areas of low strength or of deteriorated concrete, and to determine when it is safe to remove forms. For any of the uses to which both can be put, however, the Schmidt hammer has an advantage because of the larger number of tests that can be made on a given area, lower cost per test, and nondestructiveness.

SCHMIDT HAMMER

The Schmidt hammer was developed in Switzerland about 1950 (2, 3, 4) and has become a popular instrument for making rebound measurements. Essentially, the hammer measures some property of the surface layer of the concrete that has been referred to by terms such as surface hardness or coefficient of restitution, and the rebound readings are affected by the surface condition of the concrete. Although the readings are relative, some investigators have reported that, when the instrument is properly calibrated against the compressive strength of test specimens for the conditions of a particular concrete, indications of compressive strength within 15 percent can be obtained (5). The instrument can be used to check uniformity of concrete quality, to locate deteriorated areas or areas of low strength, and to determine when forms may be removed.

The Schmidt hammer consists of a steel plunger and a tension spring in a tubular frame. When the head of the device is pressed against the surface of the concrete, the hammer is retracted against the force of the spring; and when the head is completely

retracted, the spring is automatically released. The hammer is driven against the concrete and rebounds. The rebound distance is indicated by a pointer on a scale 75 mm long that is graduated from 0 to 100. The rebound readings are termed R-values. The test has to be made on a smooth spot free from honeycomb, and, if necessary, a spot is prepared by smoothing with silicon carbide stone.

For any selected area, several rebound readings may be taken, and their average may be used in estimating the compressive strength, if desired. The manufacturer furnishes a graph showing a relationship between compressive strength of the concrete and rebound readings based on data from tests conducted at the Swiss Federal Materials Testing and Experimental Institute.

WINDSOR PROBE TEST SYSTEM

The Windsor probe test system has been developed more recently. The development of the instrument began about 1964 as a joint undertaking of the Port of New York Authority and the Windsor Machinery Company of Connecticut. The device consists of a special driving unit or gun into which is inserted a hardened alloy probe that is driven into the concrete by the firing of a powder charge (Fig. 1). The manufacturer states that the penetration of the probe reflects "the precise compressive strength in a localized area."

The measurement on which the test is based is the length of probe projecting from the surface of the concrete. The lengths of individual probes may be measured by using a device supplied with the instrument, or the average of the 3 probes fired in a triangular pattern may be measured by using a mechanical averaging device also supplied. The mechanical averaging device consists of 2 triangular plates and a depth gage. One of the plates slips over the 3 probes and rests on the surface of the concrete. The other plate fits over the top of the 3 probes, and the depth gage is inserted through a hole in the center of this plate to measure a mechanical average of the exposed height of the 3 probes. The bottom plate has a $^3/_{16}$-in. circle inscribed in its center; and, if the tip of the gage rod falls outside this circle because of uneven height of the 3 probes and consequent tipping of the top plate, the measurement is rejected. This device is supposed to reject groups of 3 measurements for which the within-group coefficient of variation is greater than 3 percent.

To translate probe measurements to strength measurements, the manufacturer supplies a set of 5 calibration curves, each curve corresponding to a specified Mohs' hardness for the coarse aggregate used in the concrete.

TEST PROGRAM

The test program was designed to include the effect of the following variables: (a) type of coarse aggregate—crushed limestone, crushed traprock, river gravel, and mortar with no coarse aggregate; (b) size of coarse aggregate—1-in. to No. 4 and 2-in. to No. 4; and (c) age of curing (strength level)—3, 7, 14, and 28 days. The physical properties of the 3 coarse aggregates are as follows:

Property	Limestone	Traprock	Gravel
Absorption, percent	0.4	0.7	1.0
Bulk specific gravity (dry)	2.76	2.96	2.55
Soundness (sodium sulfate), percent loss	0.2	0.4	1.4
Los Angeles abrasion, percent loss	21	16	31

The gravel was composed of 36 percent quartz, 26 percent quartzite, 22 percent sandstone, 13 percent chert and flint, and 3 percent miscellaneous rock types.

All concrete was designed to have a cement factor of 6 ± 0.1 bag/yd^3, an air content of 6 ± 0.5 percent, and a slump of 3 ± 0.5 in. The fine aggregate fraction was 43 percent by volume of the total aggregate for all concrete containing 1-in. aggregate and 36 percent by volume in the case of concrete containing 2-in. aggregate. The water content was varied to produce the desired slump.

For each type of concrete, 6 batches were made, and from each batch one 16- by

20- by 8-in. slab and six 6- by 12-in. cylinders were cast. The slabs were consolidated by vibration and finished with a wooden float. All specimens were moist-cured in the molds for 24 hours, then removed from the molds, and stored in the moist room until the time of test.

Probe, rebound, and cylinder tests were made at 3, 7, 14, and 28 days after casting. At each age 18 individual probe tests and 9 compressive tests on cylinders were made according to the following pattern. Six probes were fired into the top and 6 probes into the bottom of 1 slab; then 3 probes were fired into the top and 3 into the bottom of a second slab. The 6 cylinders from the batch corresponding to the first slab and 3 of the cylinders from the second batch were broken at the same time. At a later age, 3 more probes were fired into the top and bottom of the second slab, which had only 3 probes, and 6 each into the top and bottom of another slab. Three companion cylinders for the second slab and all 6 cylinders accompanying the third slab were broken. The probes were fired in a triangular pattern by using the triangular plate furnished with the instrument (Fig. 1).

Before the probe and cylinder compression tests were made, Schmidt hammer tests were made on both slabs and cylinders. On the slabs, 100 rebound readings were taken: 20 each on the top and bottom of the slab, 20 around the sides near the top of the slab, 20 around the sides near the bottom of the slab, and 20 around the sides half-way between the top and bottom. On the slabs that were tested at 2 ages, 100 rebound readings were taken at each of the 2 ages. On the cylinders, 60 readings were taken: 20 near the top of the side, 20 around the middle, and 20 near the bottom. The cylinders were placed in a testing machine and subjected to a load of 10,000 lb/ft^3, just sufficient to hold them firm, while rebound tests were being made.

In this study, as in tests conducted at the National Ready Mixed Concrete Association (6), it was found that considerable deflection of the probes as well as differences in penetration often resulted from the probes striking coarse aggregate particles, making it impossible, in many cases, to use the mechanical averaging device or causing the device to fall in the reject region. Because most of these probe tests were considered to be valid tests, the individual probe measurements were used instead of averages. When the within-group coefficients of variation were calculated for all groups for which there were 3 usable probes, 96 out of 128, or exactly 75 percent, had coefficients of variation greater than 3 percent. Four measurements 90 deg apart around the probe were made with a micrometer caliper, and these 4 measurements were averaged to give the individual probe readings.

DATA

Tables 1, 2, 3, and 4 give averages, standard deviations, and coefficients of variation for the compressive strengths of the cylinders, the rebound values, and the probe measurements respectively for the different aggregates, different sizes of aggregates, and different ages. For the cylinders and probes (Tables 1 and 4), the averages are based on 9 measurements each, 6 from one batch of concrete and 3 from another. Rebound values given in Table 2 are based on 20 measurements each, all on the same slab. In each case this was the slab that was tested at only 1 age. Rebound readings on the slabs that were tested at 2 ages were not used except for comparisons among batches and to obtain figures based on 9 measurements for comparison with cylinders and probes.

Table 3 gives the figures based on those 9 rebound measurements. For this analysis, 6 rebound numbers were selected at random (by using a table of random numbers) from the 20 on the slab for which 6 probes and 6 cylinders were available. Three rebound numbers were also selected at random from the 20 on the slab for which 3 probe measurements and 3 cylinder strengths were obtained. Those 9 rebound numbers were averaged and are the basis for the averages, standard deviations, and coefficients of variation given in Table 3. The standard deviations given in Tables 1, 3, and 4 are larger than they would be if based on only single batches because of some significant differences between the 2 batches tested at the same age, but comparisons among the 3 sets of figures are on the same basis.

Figure 1. Windsor probe test device.

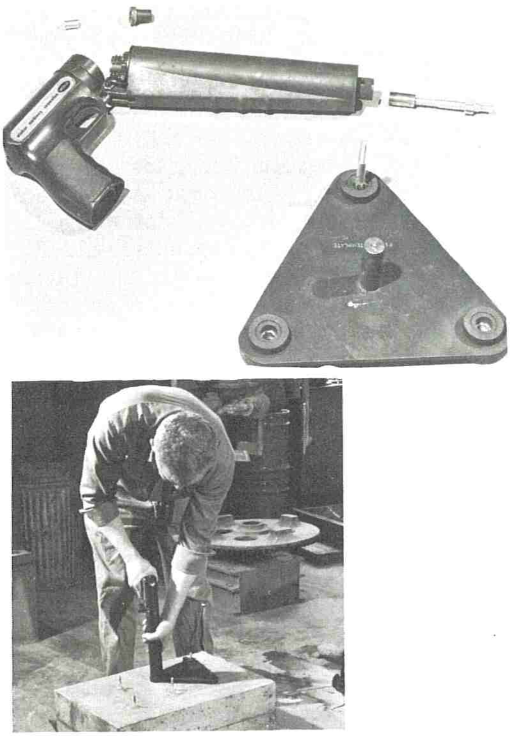


Table 1. Average compressive strength, standard deviation, and coefficient of variation for 9 cylinders.

Aggregate	Maximum Size (in.)	Age (day)	Compressive Strength (psi)	Standard Deviation (psi)	Coefficient of Variation (percent)
Gravel	1	3	3,216	31	1.0
Limestone	1	3	2,953	282	9.6
Traprock	1	3	3,031	66	2.2
Gravel	1	7	4,252	134	3.1
Limestone	1	7	4,441	168	3.8
Traprock	1	7	4,422	76	1.7
Limestone	1	14	5,096	91	1.8
Traprock	1	14	5,240	63	1.2
Gravel	1	28	5,462	71	1.3
Limestone	1	28	6,040	100	1.7
Traprock	1	28	5,689	70	1.2
Limestone	2	3	3,101	38	1.2
Traprock	2	3	2,950	62	2.1
Limestone	2	7	4,354	91	2.1
Traprock	2	7	3,879	96	2.5
Limestone	2	14	4,797	141	2.9
Traprock	2	14	4,371	124	2.7
Limestone	2	28	5,476	169	3.1
Traprock	2	28	5,108	130	2.5
Mortar	-	7	4,343	155	3.6
Mortar	-	14	4,834	100	2.1
Mortar	-	28	5,614	107	1.9

Table 2. Average value, standard deviation, and coefficient of variation for 20 rebound measurements.

Aggregate	Maximum Size (in.)	Age (day)	Side of Slab	R-Value	Standard Deviation	Coefficient of Variation (percent)
Gravel	1	3	Top	25.4	1.8	7.0
			Bottom	28.5	3.2	11.2
Limestone	1	3	Top	23.5	1.8	7.6
			Bottom	26.6	3.4	12.8
Traprock	1	3	Top	20.7	2.4	11.7
			Bottom	25.4	3.5	13.9
Gravel	1	7	Top	28.3	2.2	7.6
			Bottom	31.0	1.7	5.4
Limestone	1	7	Top	29.0	2.1	7.2
			Bottom	31.7	2.7	8.4
Traprock	1	7	Top	25.3	2.8	10.9
			Bottom	29.0	2.5	8.5
Limestone	1	14	Top	28.3	2.8	10.1
			Bottom	33.5	2.7	8.1
Traprock	1	14	Top	28.0	1.7	6.1
			Bottom	32.6	3.1	9.6
Gravel	1	28	Top	30.2	2.5	8.3
			Bottom	37.1	3.6	9.8
Limestone	1	28	Top	31.2	1.6	5.2
			Bottom	36.4	4.3	11.8
Traprock	1	28	Top	27.4	2.0	7.2
			Bottom	34.1	4.3	12.5
Limestone	2	3	Top	24.4	2.6	10.8
			Bottom	28.4	5.1	17.9
Traprock	2	3	Top	20.3	1.9	9.2
			Bottom	22.6	2.8	12.3
Limestone	2	7	Top	25.0	2.3	9.2
			Bottom	29.2	3.4	11.8
Traprock	2	7	Top	24.0	1.7	6.9
			Bottom	27.4	4.8	17.6
Limestone	2	14	Top	27.2	2.3	8.4
			Bottom	32.6	4.6	14.0
Traprock	2	14	Top	23.8	2.8	11.6
			Bottom	26.8	3.3	12.4
Limestone	2	28	Top	29.4	1.6	5.5
			Bottom	33.4	2.3	7.0
Traprock	2	28	Top	25.3	2.2	8.6
			Bottom	31.1	3.2	10.2
Mortar		7	Top	26.0	1.7	6.7
			Bottom	27.6	1.2	4.3
Mortar		14	Top	25.5	1.7	6.9
			Bottom	29.8	0.8	2.8
Mortar		28	Top	26.8	2.5	9.4
			Bottom	32.4	1.7	5.2

Table 3. Average value, standard deviation, and coefficient of variation for 9 rebound measurements.

Aggregate	Maximum Size (in.)	Age (day)	Side of Slab	R-Value	Standard Deviation	Coefficient of Variation (percent)
Gravel	1	3	Top	24.8	2.0	8.0
			Bottom	28.2	2.4	8.6
Limestone	1	3	Top	25.6	3.2	12.3
			Bottom	26.6	4.0	15.1
Traprock	1	3	Top	20.4	2.7	13.0
			Bottom	25.6	2.7	10.5
Gravel	1	7	Top	28.2	1.8	6.3
			Bottom	31.2	1.6	4.9
Limestone	1	7	Top	28.2	2.3	8.1
			Bottom	31.4	2.6	8.3
Traprock	1	7	Top	25.0	1.9	7.5
			Bottom	29.7	4.6	15.5
Limestone	1	14	Top	29.4	1.3	4.5
			Bottom	34.0	2.0	5.9
Traprock	1	14	Top	28.0	1.7	5.9
			Bottom	32.9	2.0	6.2
Gravel	1	28	Top	31.0	2.8	9.0
			Bottom	37.0	3.0	8.2
Limestone	1	28	Top	31.0	1.9	6.2
			Bottom	36.8	3.7	10.1
Traprock	1	28	Top	28.3	3.2	11.2
			Bottom	35.6	5.2	14.5
Limestone	2	3	Top	23.3	1.9	8.0
			Bottom	27.0	7.0	26.0
Traprock	2	3	Top	19.9	2.4	12.1
			Bottom	27.9	3.1	13.4
Limestone	2	7	Top	25.8	1.8	6.9
			Bottom	29.8	2.9	9.9
Traprock	2	7	Top	23.8	2.9	12.2
			Bottom	26.3	3.2	12.3
Limestone	2	14	Top	28.0	2.2	8.0
			Bottom	33.2	5.7	17.0
Traprock	2	14	Top	24.7	2.8	11.2
			Bottom	28.1	4.4	15.8
Limestone	2	28	Top	29.2	2.4	8.4
			Bottom	33.1	3.4	10.4
Traprock	2	28	Top	25.6	2.0	7.9
			Bottom	30.7	3.1	10.2
Mortar		7	Top	25.3	1.9	7.7
			Bottom	27.9	1.2	4.2
Mortar		14	Top	26.0	1.5	5.8
			Bottom	30.4	0.9	2.9
Mortar		28	Top	27.4	3.2	11.7
			Bottom	31.9	2.3	7.1

Table 4. Average penetration, standard deviation, and coefficient of variation for probe measurements.

Aggregate	Maximum Size (in.)	Age (day)	Side of Slab	Number of Probes	Penetration (in.)	Standard Deviation (in.)	Coefficient of Variation (percent)
Gravel	1	3	Top	9	1.730	0.171	9.9
			Bottom	9	1.871	0.153	8.2
Limestone	1	3	Top	9	1.564	0.054	3.4
			Bottom	9	1.631	0.106	6.5
Traprock	1	3	Top	9	1.622	0.102	6.3
			Bottom	9	1.835	0.080	4.4
Gravel	1	7	Top	9	1.814	0.084	4.6
			Bottom	9	1.955	0.102	5.3
Limestone	1	7	Top	9	1.797	0.058	3.2
			Bottom	9	1.829	0.114	6.2
Traprock	1	7	Top	9	1.849	0.151	8.2
			Bottom	9	1.964	0.094	4.8
Limestone	1	14	Top	9	1.852	0.113	6.1
			Bottom	9	2.012	0.112	5.6
Traprock	1	14	Top	9	1.825	0.073	4.0
			Bottom	9	2.098	0.076	3.6
Gravel	1	28	Top	9	1.987	0.166	5.8
			Bottom	9	2.086	0.069	3.3
Limestone	1	28	Top	9	1.923	0.126	6.6
			Bottom	9	2.034	0.082	4.0
Traprock	1	28	Top	9	1.965	0.115	5.9
			Bottom	9	2.064	0.048	2.3
Limestone	2	3	Top	9	1.728	0.260	15.1
			Bottom	9	1.858	0.100	5.4
Traprock	2	3	Top	9	1.694	0.147	8.7
			Bottom	9	1.906	0.124	6.5
Limestone	2	7	Top	9	1.906	0.073	3.8
			Bottom	9	2.008	0.130	6.5
Traprock	2	7	Top	7	1.807	0.134	7.4
			Bottom	6	1.843	0.113	6.1
Limestone	2	14	Top	9	1.894	0.107	5.7
			Bottom	9	2.027	0.131	6.4
Traprock	2	14	Top	8	1.972	0.159	8.1
			Bottom	8	1.966	0.137	7.0
Limestone	2	28	Top	9	2.049	0.124	6.1
			Bottom	8	2.007	0.114	5.7
Traprock	2	28	Top	9	2.005	0.192	9.6
			Bottom	9	2.150	0.133	6.2
Mortar		7	Top	9	1.512	0.108	7.1
			Bottom	9	1.657	0.061	3.7
Mortar		14	Top	9	1.595	0.101	6.3
			Bottom	9	1.802	0.030	1.7
Mortar		28	Top	9	1.701	0.071	4.2
			Bottom	9	1.844	0.047	2.5

For the gravel aggregate, not enough of the larger sizes was available and concretes were made only with the 1-in. top size. Also no data are given for the gravel aggregate at the age of 14 days. Considerable difficulty was experienced with the probe tests for this concrete at this age. Many of the probes bounced out or broke and had to be reshot. Only 11 measurable probes were finally obtained, and there was doubt as to their validity. The rebound and cylinder data could have been used, but it was decided to eliminate all data for the gravel concrete at 14 days.

No data are given for mortar at 3 days. There was difficulty with the probes penetrating too deeply. We made an effort to overcome this difficulty by air-drying the slabs and cylinders. Thus, because the treatment of the 3-day mortar slabs and cylinders was different from that of the concrete and of the mortar specimens at later ages, the 3-day mortar data were all eliminated.

Data are given in Tables 2, 3, and 4 for both top and bottom of the slabs because there were significant differences between top and bottom for both rebound and probe measurements. For compressive strengths of cylinders, of course, there is no differentiation between top and bottom, although rebound measurements taken on the sides of the cylinders showed higher readings near the bottom than near the top.

A column for the number of probes is given in Table 4 because there were a few cases with the 2-in. aggregate where 9 usable probes were not available. Frequently probes would break or bounce out and have to be reshot. The triangular device spaces the probes approximately 7 in. apart, and individual probes cannot be driven closer without danger of interfering with one another. When a probe fails, the usual procedure is to turn the triangular plate around, fit it over the good probes, and fire another one on the other side of the 2 good ones from the unusable one. In some cases the result was that 9 measurable probes were not obtained. In one case, a single probe that was measured and recorded was so far from the other two that it was eliminated.

All of the tests in this investigation were conducted in the laboratory, and the cylinder strengths were quite uniform. With the exception of the 1-in. limestone at 3 days, all of the calculated standard deviations were under 200 psi and the coefficients of variation were under 4 percent. In fact, 17 out of the 22 coefficients of variation were under 3 percent, and that represents excellent laboratory control according to ACI 214. A pooled value for the standard deviation, excluding the 3-day, 1-in. limestone, was 107 psi based on 189 cylinders.

The large standard deviation for the 1-in. limestone at 3 days was produced by the fact that the 3 cylinders from the second batch of concrete all tested significantly higher than the 6 from the first batch. The averages were 3,323 psi with a coefficient of variation of 2.3 percent and 2,768 psi with a coefficient of variation of 1.4 percent respectively.

The data given in Table 2 based on 20 rebound measurements indicate that the presence of coarse aggregate in the mixes caused more scatter in the results. The pooled standard deviation for the mortar mixes was 1.7 based on 54 rebound numbers; that for the concretes was 2.9 based on 342 rebound numbers. Coefficients of variation for all groups ranged from 2.8 to 17.9 percent.

The probe data (Table 4) indicate that the standard deviations were relatively constant over all the conditions within a given size of aggregate. The standard deviations were 0.143 based on 136 probes for the 2-in. top size, 0.105 based on 198 probes for the 1-in. top size, and 0.075 based on 54 probes for the mortar. Coefficients of variation ranged from 1.7 to 15.1 percent.

Comparison of the coefficients of variations given in the tables shows that the coefficients for the probe measurements were generally lower than those for rebound numbers, whether the latter were based on 9 or 20 measurements. It appears that the use of 20 measurements from 1 slab or 9 measurements from 2 slabs did not make any difference in the coefficients of variation for rebound numbers. Exactly half (22 out of 44) of the coefficients for the 9-probe averages were numerically equal to or larger than those for the 20-probe averages. The coefficients for the cylinders (Table 1) were generally lower than those for either the probe measurements or rebound numbers.

COMPARISONS OF BATCHES

Table 5 gives a summary of the results of comparisons of different concretes. Comparisons are based on the t-test for significant difference between averages (7) at the 95 percent confidence level.

There were significant differences between the concrete in the 2 batches, as indicated by the cylinders, that were not revealed by either the rebound or the probe measurements. The most consistent differences, based on cylinder strengths (Table 5), were for the 3-day concrete for all 3 aggregates with 1-in. top size and for the mortar specimen. For the rebound readings, there were only 3 significant differences out of 44 possible when 9 readings were used, and the 10 did not in general correspond with differences shown by the cylinders. For the probes, only 1 out of 42 possible differences showed a t-value beyond 95 percent, and this is no more than would be expected by chance.

COMPARISONS OF AGGREGATES

All comparisons of cylinders except two were highly significant. The 2 exceptions are limestone versus traprock at the 2 earliest ages. The direction of the differences in this instance is interesting. For comparisons between the gravel and the other 2 aggregates, the gravel concretes were the stronger at 3 days; but both the limestone and traprock showed higher strengths than the gravel at 7 and 28 days. For comparisons of limestone versus traprock with 1-in. top size, there was no significant difference for the 2 earlier ages; the traprock showed higher strength at 14 days, and the limestone showed higher strength at 28 days. For the 2-in. top size, however, the limestone showed significantly higher strength than the traprock at all 4 ages.

For the rebound numbers, 17 out of the 28 comparisons showed significant differences. Of these differences, 10 were on the top sides of the slabs and 7 were on the bottom. For the probe measurements, there were only 7 significant differences out of a possible 28. Neither the hammer nor the probes consistently showed the differences in the same direction as those shown by the cylinder averages. The rebound numbers agreed with the cylinders in showing the 2-in. limestone concrete to have higher values than those of the traprock with the same top size at 3, 14, and 28 days, and the gravel higher than the traprock at 3 days. However, they disagreed with the cylinders in showing higher values for the 1-in. gravel than for the traprock at 7 and 28 days. The probes agreed with the others as to significance and direction of the difference in only 1 case, and 6 of the 7 indicated significant differences were on the bottom of the slabs.

COMPARISONS OF 1- AND 2-IN. AGGREGATE

In comparisons of concretes made with the 2 different maximum sizes, significant differences shown by the cylinders were not shown so consistently by either rebound or probe measurements. There were 6 out of a possible 8 significant differences, the 2 exceptions being for the limestone at the 2 earlier ages. In all cases of significant difference, the concrete without the larger size fraction showed greater strength.

The rebound numbers agreed with the cylinder strengths in all cases where the difference was significant for both. Differences for the rebound numbers were significant on both sides of the slab, the 1-in. size showing higher readings for the limestone at 7 and 28 days and for the traprock at 14 and 28 days.

However, the probe measurements were contradictory to the other two. Of the 7 cases where the difference was significant based on the probe measurements, 5 showed less penetration for the 2-in. aggregate than for the 1-in. This is undoubtedly related to the increased number of instances of probes striking large pieces of aggregate.

COMPARISONS OF SUCCESSIVE AGES

Differences among averages for successive ages of test showed increases of strength with age for all 3 types of test; but for the rebound and probe measurements, there were cases where there was no significant difference. For the rebound numbers, 11 out of the 30 differences were not significant. For the probes, 19 out of 30 differences were not significant.

Table 5. Number of comparisons having significant differences at 95 percent confidence level.

					Rebound			
	Cylinders		Probes		9 Values		20 Values	
Item Compared	Number	Differences	Number	Differences	Number	Differences	Number	Differences
Batches	22	9	43	1	44	3	44	10
Top versus bottom	—	—	22	12	22	15	22	22
Aggregates	14	12	28	7	28	12	28	17
1 in. versus 2 in.	8	6	16	7	16	7	16	9
Successive ages								
3 to 7 days	5	5	10	4	10	8	10	8
7 to 14 days	5	5	8	3	10	4	10	6
14 to 28 days	5	5	10	4	10	0	10	5
Total	59	42	137	38	140	39	140	77

Figure 2. Regression curve and confidence limits for rebound values.

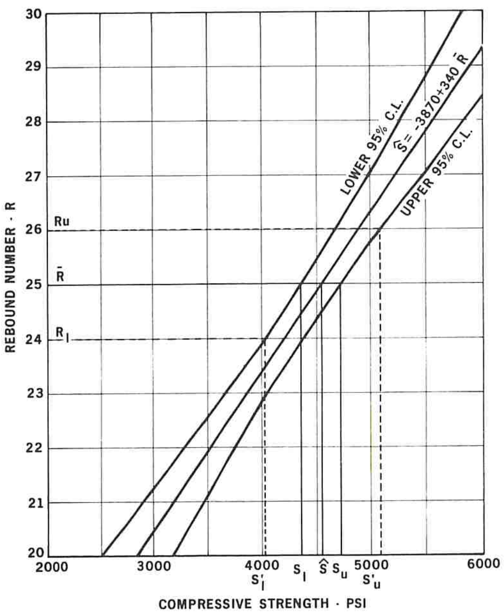
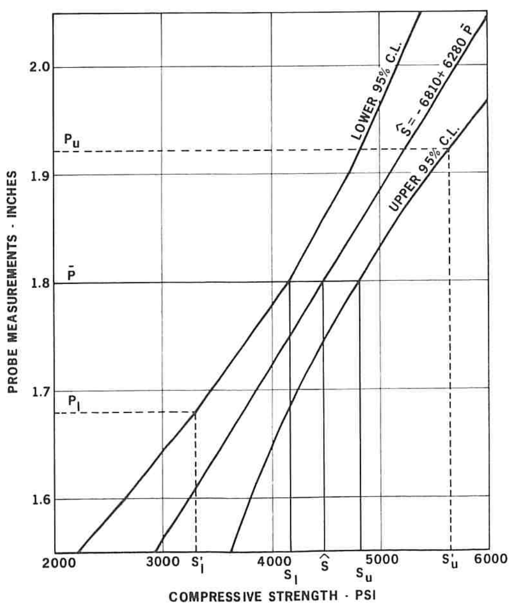


Figure 3. Regression curve and confidence limits for probe measurements.



COMPARISONS OF TOP AND BOTTOM OF SLABS

Comparisons between measurements on the top and on the bottom of the same slab showed a highly significant difference for rebound numbers in every case, with the readings being higher on the bottom than on the top. For the probes, no significant difference was detected in 10 out of the 22 cases. The rebound tests also showed significant differences between top and bottom of cylinders.

SUMMARY OF SIGNIFICANT DIFFERENCES

Table 5 gives the number of significant differences for each of the 3 test methods. Rebound numbers are also given for 9 measurements selected at random from the 40 measurements on the appropriate slab. Not all of the differences investigated are expected to be significant, but the significant differences shown by the compressive tests on the cylinders may be taken as representative of differences in strength that actually existed and used as a criterion with which to compare results of the other 2 methods of test, excluding comparisons between top and bottom, which were not available for the cylinders. The percentage of comparisons that were significant is as follows:

Test Method	Percent
Cylinders	71
Probes	23
Rebound	
9 values	39
20 values	47

PREDICTION OF COMPRESSIVE STRENGTH FROM REBOUND MEASUREMENTS

Figure 2 shows a regression curve and confidence limits for compressive strengths versus rebound numbers. The confidence limits are 95 percent limits for the location of the fitted line, calculated in the manner described by Natrella (7, Ch. 5).

The confidence interval for the line as a whole is used instead of the confidence interval for points on the line because the line is a calibration line that will be used repeatedly for prediction purposes. Natrella gives a further discussion of this point (7, pp. 5-15 to 5-16).

When a regression curve is plotted or calculated with 2 sets of measurements such as this, 2 conditions must be met: The data used in the relationship must fit a common regression line, and the precision (that is, the scatter about the line) must be about the same for all the sets of data used. For the rebound numbers, as for the probe measurements, the largest available group of data that fulfilled these conditions was used. Also, because no differentiation between sides was possible for cylinders and usually only 1 side of a concrete slab is accessible, data from the top sides only were used.

Data shown in Figure 2 are based on the measurements of the slabs and cylinders made with traprock aggregate with both top sizes. For both the rebound measurements and the cylinders, the standard deviations were reasonably uniform over the range of strengths and conditions. Also, the individual regression lines for the 1- and 2-in. traprock were sufficiently close together so that the data could be plotted together and 1 regression line drawn.

The regression line is based on 16 plotted points obtained as follows: For each age for each type of concrete (1- or 2-in. aggregate), 2 values for rebound measurements were obtained by averaging the 20 rebound numbers on the top side of each of the 2 slabs tested at that age. These values were paired with the averages of 3 cylinder strengths from the corresponding batches of concrete. Three cylinders were used because only 3 were available from the batches for which the slabs were tested at 2 ages. For the batches that were tested at only 1 age, 3 of the 6 cylinders were selected at random for averaging.

Figure 2 also shows the 95 percent confidence band for the location of the fitted line and illustrates the use of the line predicting compressive strength of the same type of concrete made with the same materials. If an average rebound reading, $\bar{R}$ in Figure 3,

is 25, the horizontal line from $\bar{R}$ intersects the prediction line at a point indicating an average compressive strength, S, of 4,530 psi. The confidence interval, assuming no error in the rebound average, is from S_1 = 4,350 psi to S_u = 4,710 psi, a range of 360 psi.

To calculate a confidence band for the rebound measurements, we obtained an estimate of the standard deviation of individual rebound numbers from the 16 sets of 20 measurements obtained on the top side of the slabs made with traprock. This estimate obtained was 2.23, based on 320 measurements. Dividing by 20 to give the standard deviation of averages of 20 and multiplying by ±1.96 to give the 95 percent confidence limits give approximately ±1.0 for the confidence band. The horizontal lines from $\bar{R}_1$ = 24 and $\bar{R}_u$ = 26 intersect the confidence limits for points on the line at S_1' = 4,010 psi and S_u' = 5,070 psi, a range of 1,060 psi. The combined probabilities give a confidence of approximately 90 percent for this range.

PREDICTION OF COMPRESSIVE STRENGTH FROM PROBE MEASUREMENTS

Figure 3 shows the results of corresponding calculations of the regression line and confidence limits for the probe measurements. In this case the individual regression lines for all 3 aggregates for the 1-in. size were sufficiently close together so that 1 regression line could be fitted to all 3 sets of data. The averages of the 3 probe measurements that were fired into the slabs tested at 2 ages and the averages of 3 selected at random from the 6 fired into the slabs tested at only 1 age were paired with the corresponding averages of 3 cylinders to give 22 pairs of values for plotting and calculating the line.

The confidence band for the average of probes was obtained from an estimate of the standard deviation from the 99 probes fired into the top of the slabs made with the 1-in. aggregate; within-batch deviations were used. This estimate was 0.109 in. Dividing by 3 to get the standard deviation of averages of 3 and multiplying by ±1.96 give ±0.12 in. for the 95 percent confidence band. A value of 1.8 for the average of 3 probes gives $\hat{S}$ = 4,490 psi for the estimated strength. The final confidence band for the strength resulting from both the confidence band for points on the line and the confidence band for the probe measurements is from S_1' = 3,290 psi to S_u' = 5,650 psi, a range of 2,360 psi.

PREDICTION OF STRENGTHS USING MANUFACTURERS' CURVES

The Schmidt hammer and the Windsor probe system are both supplied with curves or tables showing a relation between compressive strength and rebound number or inches of exposed probe respectively. The curve supplied with the Schmidt hammer was calculated from tests made by the Swiss Federal Materials Testing and Experimental Institute on approximately 550 concrete cubes; the values are reduced by 10 percent to compensate for the higher strength results obtained from cubes. The manufacturers of the Windsor probe system provide a set of curves based on different Mohs' hardness of the aggregates. No information on the source, character, or number of tests on which these curves are based is given, and no limits of uncertainty are indicated. Information supplied with the Schmidt hammer states that the curve supplied is based on "average" concrete and conditions, but for any sizable application it is advisable to develop a new calibration curve for the particular conditions on the job. In the NRMCA study mentioned previously (6) and in our tests, manufacturer's curves for the Schmidt hammer were not used.

For the Windsor probe system, NRMCA found that its laboratory curves fit the data much better than did the manufacturer's curves. In a study conducted by the Louisiana Department of Highways (8) in which the chert aggregate used was considered to have a Mohs' hardness of 7, it was found that the manufacturers' curve would have caused a considerable overestimation of the strength, compared to that obtained on the cylinders. In the current study, there is some doubt about the appropriate number for Mohs' hardness, but none of the manufacturer's curves fitted the data as well as the lines calculated from the data.

MEASUREMENTS REQUIRED TO DETECT DIFFERENCES IN STRENGTH

Assessment of the effectiveness of different methods of measuring strength may be made by determining the number of specimens required to detect the true average strength within given limits with a given probability. The following data are based on the requirement of detecting a difference from true strength of 200 psi with an α error of 0.10 and a β error of 0.10. This means that the averages of a group of tests would show a significant difference from an assumed true strength 1 time in 10 when the actual difference is 0 psi and 9 times in 10 when the actual difference is 200 psi or greater. The number of measurements required depends on the standard deviation of the measurements involved.

For the cylinders in this investigation, all the standard deviations for aggregate-age combinations except one were less than 200 psi (Table 1). These were carefully prepared, well-cured, laboratory cylinders. Two hundred psi represents a coefficient of variation of 5 percent for 4,000-psi concrete. If 200 psi is taken as a standard deviation for cylinders and the α and β errors described earlier are used, approximately 8 cylinders would be required to detect the true strength within 200 psi with a probability of 90 percent. The sample sizes given in this section are taken from a curve based on Table A-12C of Dixon and Massey (9).

To determine the number of measurements required to detect a psi difference for rebound and probe measurements, one has to take into account not only the standard deviation of the measurements but also the slope of the calibration curve used to convert the measurements to psi. For our rebound measurements, the slope of the calibration curve (Fig. 2) is 336 psi per rebound number. The published manufacturer's curve, although not linear, is very nearly so in the middle range and has a slope of approximately 200 psi per rebound number in the range from about 3,000 to 5,000 psi. Using 340 for the slope and 2.2 for the standard deviation of rebound numbers (as given earlier for our data) would require about 120 rebound measurements to detect the average strength within 200 psi. For tests for which the slope of the calibration curve is 200, as given by the manufacturer's curve, the number would be about 50.

For the probe measurements, the slope of the line for 1-in. aggregate shown in Figure 3 is 6,280 psi per inch of probe. Using 6,300 for the slope and 0.1-in. for the standard deviation would require about 85 probes to detect the average strength within 200 psi. The manufacturer's curves have slopes that range from 6,750 psi per inch of probe for Mohs' hardness 3 to 8,700 for 7. For data from concretes that fit these calibration curves, the approximate number of measurements required to determine the strength within 200 psi would be 100 for Mohs' hardness 3 and 160 for 7.

CONCLUSIONS

The following conclusions are based on analysis of the probe, rebound, and cylinder data from this study.

1. Coefficients of variation for both rebound and probe measurements were generally higher than those obtained from compression tests of companion cylinders.
2. Coefficients of variation for probe measurements were generally lower than those for rebound numbers.
3. The variance of rebound numbers was increased by the presence of coarse aggregate. The standard deviation was 1.7 for the mortar tests and 2.9 based on data from all the concretes.
4. For probe tests, the standard deviations were relatively constant for a given size of aggregate: 0.75 in. for mortar, 0.105 in. for all tests for 1-in. top size, and 0.143 in. for 2-in. top size.
5. Significant differences between batches of the same concrete tested at the same age were shown by the cylinders and to some extent by the rebound numbers but were not shown by the probe measurements. The number of significant differences between batches revealed by rebound numbers increased when 20 instead of 9 measurements per slab were used.

6. Significant differences between concretes made with different aggregates were shown by the cylinder tests except for limestone versus traprock at 3 and 7 days. For the rebound numbers, 17 out of 28 of the comparisons of aggregates showed a significant difference; and, for the probe measurements, only 7 out of 28 were significant. In a number of cases where differences were significant for both the cylinders and one of the other methods of test, there was a disagreement on the direction of the difference.

7. For comparisons between 1- and 2-in. aggregates, the cylinders showed significant differences for all cases except the limestone at 3 and 7 days, and the rebound numbers showed significant differences except at 3 and 14 days for the limestone and at 7 days for the traprock. In both cases, all significant differences indicated higher strength for the 1-in. maximum size. The probe measurements showed no significant differences on either side of the slabs for the limestone at 14 days and the traprock at 3 and 28 days and on one side of the slabs for the limestone at 3 and 28 days and the traprock at 7 days. However, of the 7 significant differences shown by probe measurements, 5 indicated higher strength for the 2-in. maximum size.

8. All 3 methods of test showed significant increases of strength at successive ages. However, significant differences were not shown in 11 out of 30 cases for the rebound numbers and 17 out of 28 for the probes.

9. Both rebound and probe measurements showed significant differences between top and bottom of the slabs, with the bottom being stronger. The rebound numbers showed highly significant differences in every case, but 10 out of the 22 comparisons were not significant with the probes.

10. Comparisons of the numbers of significant differences detected by the 3 methods for the 7 types of comparisons show that comparisons of the averages of 9 cylinders in every case detected more significant differences than did either the rebound or probe measurements; comparisons using the averages of 9 rebound numbers did as well as or better than comparisons using 9 probes in detecting significant differences in 6 out of the 7 types of comparison; and comparisons using the average of 20 rebound numbers did better than comparisons using 9 probes in all cases.

11. The calibration curve for rebound numbers based on the data for traprock indicated that an average rebound number of 25 obtained from 20 measurements corresponded to approximately 4,530 ± 530 psi with a 90 percent confidence.

12. The calibration curve for probe measurements based on the data for the 1-in. maximum size for all 3 aggregates indicated that an average probe measurement of 1.8 in. obtained from 3 measurements corresponded to approximately 4,490 ± 1,180 psi with a 90 percent confidence.

13. To use either the Schmidt hammer or the Windsor probe system to predict compressive strength, one should develop calibration curves by using the materials and mix designs to be used on the job. Curves supplied by the manufacturers cannot be relied on.

14. Confidence limits for points on the curve calculated from the calibration data should be placed about the curves, and variation of the basic measurements (rebound numbers or probe measurements) together with the confidence interval should be considered when the curves are used.

15. If curves based on an error of 10 percent (probability of making the error of assessing a difference when none exists) are used, the number of tests required to detect the true strength of concrete within 200 psi 90 percent of the time is as follows: 8 cylinders, assuming a standard deviation of 200 psi; 120 rebound measurements, assuming a standard deviation of 2.2 and a slope of the calibration curve of 240 psi per rebound (a calibration curve with a slope of 200 psi such as that published from the manufacturer's data would require about 50 rebound measurements); and 85 probe measurements, assuming a standard deviation of 0.1 in. and a slope of the calibration curve of 6,300 psi per inch of probe (for the manufacturer's curves with slopes ranging from 6,750 to 8,700 psi per inch, 100 to 160 probes would be needed).

16. Considerable difficulty was experienced in using the mechanical averaging device supplied with the Windsor probe test system. Best results are obtained by measuring all probes individually.

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